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PHMC Post-NPH Emergency Response Training

T.J. Conrads

Numatec Hanford Company, Richland, WA 99352

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Abstract: This document describes post-Natural Phenomena Hazard (NPH) emergency response training that was provided to two teams of Project Hanford Management Contractors (PHMC) staff that will be used to assess potential structural damage that may occur as a result of a significant natural phenomena event. This training supports recent plans and procedures to use trained staff to inspect structures following an NPH event on the Hanford Site.

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PHMC Post-NPH Emergency Response Training

HN-SD-GN-ER-505

Prepared For:

U.S. Department of Energy
Richland Operations Office
Richland, Washington

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Approved by
Thomas J. Conrads, Numatec Hanford Company

Executive Summary

This report documents training that was conducted for the Project Hanford Management Contractor (PHMC) staff in compliance with DOE Order 5480.28, *Natural Phenomena Hazards Mitigation*, Section 10.d.(2), which requires procedures for inspection and mitigation of damage following a Natural Phenomena Hazard (NPH) event at U.S. Department of Energy (DOE), Richland Operations Office (RL).

The scope of training addressed procedures to identify and inspect structures and facilities at the Hanford Site that would potentially be damaged from a significant NPH event. Two separate levels of training were provided.

- ◆ Rapid Evaluation Inspectors
- ◆ Detailed Evaluators

Since the program is in its infancy, the following steps are recommended:

- ◆ Provide additional training sessions
- ◆ Update site and facility emergency plans
- ◆ Maintain training awareness
- ◆ Establish contacts with other DOE sites

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Background

In 1993, DOE-RL transmitted DOE Order 5480.28, *Natural Phenomena Hazards Mitigation*, to Westinghouse Hanford Company (WHC) for compliance. Currently the PHMC is implementing the order according to WHC-SP-1175, "Westinghouse Hanford Company Implementation Plan for DOE Order 5480.28, *Natural Phenomena Hazards Mitigation*." Requirements for the post-NPH inspection of buildings for damage, the subject of this report, are contained in paragraph 10.d.(2) of DOE Order 5480.28:

10.d.(2) Post-NPH Procedures. Facilities or sites which have SSCs in PC3 or PC4 shall have procedures to inspect the facility for damage following an NPH event, to place the facility into a safe configuration when damage occurs, and to document and report such damage.

1.0 Introduction

Existing elements of post-NPH preparedness for the PHMC are provided by the following existing reports:

- ◆ WHC-SD-GN-ER-504, Rev. 0, *Hanford Site Post-NPH Building Inspection Plan*.
This document: 1) provides procedures and site data that may be used to prioritize the inspection of potentially damaged structures and facilities, 2) provides an overview of procedures for inspection, tagging, and reporting of findings of post-NPH inspections, and 3) defines the skill requirements of individuals who would conduct inspections.
- ◆ WHC-SD-GN-ER-503, Rev. 0, *Procedure for Post-NPH Inspection of Buildings*.
This document provides detailed procedures for inspection, tagging, and reporting of findings of post-NPH inspections.

This report documents site training that was conducted by EQE International and DynCorp Tri-Cities Services, Inc. Emergency Preparedness to present the material contained in these procedures and other consensus industry standards to PHMC staff who will be responsible for the conduct of post-NPH damage investigations at the Hanford Site. Two separate levels of training were provided; first, for individuals who would act as Rapid Evaluation Inspectors (REI) for initial damage screening, and second, for individuals who would act as Detailed Evaluators (DE) for rapid, yet more comprehensive damage evaluations. Rapid Evaluation Inspector training was conducted on March 18 and 19, 1997. Detailed Evaluator training included attendance at REI training, plus additional training that was held on March 21, 1997.

2.0 Discussion and Recommendations

Upon receipt of DOE Order 5480.28, *Natural Phenomena Hazards Mitigation*, from DOE-RL and following the release of the last applicable standard to the order, the PHMC is in the midst of compliance following the commitments in WHC-SP-1175, Rev. 0, Westinghouse Hanford Company Implementation Plan for DOE Order 5480.28. The following narrative describes the training of PHMC staff for the inspection of structures following a significant natural phenomena event. This training satisfies one of the commitments in the implementation plan and requirements in the DOE Order. Since this was an initial effort to establish a group of qualified structural evaluators for the Site, a number of recommendations are also proposed to ensure the viability of this resource in the future.

2.1 Discussion

The training of staff to ensure a viable resource to assess the damage of structures following a significant natural phenomena event focuses on two groups of inspectors. One group of inspectors make up the Rapid Evaluation Team. This team is made up of members of various facilities and is intended to be the initial source of information on structural damage to the Building Emergency Director. They will also placard the structure as safe, limited entry, or unsafe.

The other group of inspectors are more of a PHMC wide resource to provide follow-on final detailed assessment of the questionable structures and disposition them accordingly. This team is called the Detailed Evaluation Team.

2.1.1 Rapid Evaluation Inspector (REI) Training

The schedule of training and list of attendees for the REI training session are presented Appendices A and D, respectively. Also presented in Appendices B and C are copies of the overhead transparencies which were utilized during the training. Not included in the Appendices are a few overheads which described the history of past seismic events in the general Hanford vicinity, geologic sources of possible future seismic events, and a general summary of seismic risk studies that have been developed for the Hanford Site.

One overhead shown in Appendix B was revised during the presentation. The overhead which is titled "Posting Classification System" and presents conditions for Changing of Posting, was revised during a class group discussion when it was concluded that it would be cumbersome to impose a requirement for changing the building posting that was developed for controlling the facility lock and tag configuration. A revision was presented for developing a logging system that would follow similar guidelines to the lock and tag standard, but would be somewhat more flexible.

Also presented, but not included in the Appendices, were a series of about 110 slides which presented general aspects of structural behavior during seismic events, and photographs of geologic/foundation related damage, as well as shaking - induced damage for various types of construction. Damage photographs were presented for degrees of damage which would normally be associated with unsafe, or "red tag", conditions as well as for limited entry, or "yellow tag", conditions.

All attendees were given a brief test of course content at the conclusion of the training, see Appendix F. The test consisted of 5 open book, multiple choice questions, plus a "practical" examination in which examinees were asked to judge damage conditions from a series of 6 damage photographs with attached brief "scenario" descriptions, with possible answers given in multiple choice format. Average scores for the open book and practical sections were 92% and 76%, respectively, with a net average score of 83%. High and low net scores were 95% and 64%, with more than 90% of the examinees achieving a score greater than 70%.

2.1.2 Detailed Evaluator (DE) Training

The schedule of training and list of attendees for the DE training session are shown in Appendices A and E, respectively. Appendix C includes copies of the overhead transparencies that were presented during the training, as well as, additional "handout" technical materials.

All Detailed Evaluator trainees were required to attend both the REI and the DE training. Thus, the DE training was a supplementary session which went into somewhat greater technical detail regarding engineering aspects of evaluating the in-situ strength of existing damaged structures. Presented material included:

- ◆ The Detailed Evaluation Form and procedures presented in WHC-SD-GN-ER-503, Rev. 0 *Guidelines for Estimating the In-Situ Strengths of Damaged Reinforced Concrete or Masonry Walls*.
- ◆ A brief presentation of possible sources of cracking-related damage to steel frame construction observed following the Northridge earthquake.
- ◆ Inspection and repair guidelines from FEMA 267.

At the conclusion of the training session, a brief examination was given consisting of 6 multiple-choice questions, see Appendix G. The question topics varied from relatively simple to subjects requiring a relatively good understanding of structural engineering principles. The resulting average test score achieved was 89%. High and low scores were 100% (8 examinees) and 80% (3 examinees).

2.2 Recommendations

Since this was an initial effort to provide post-NPH emergency response inspection training to PHMC staff, a few recommendations are provided to ensure a viable resource in the future.

- ◆ Determine what additional training sessions that are necessary to allow trained structural safety inspection personnel limited access to all major facilities.
- ◆ Update site and facility emergency plans to include post-NPH structural safety inspections in accordance with established procedures.
- ◆ Include structural safety inspection personnel during future site emergency training exercises to maintain training awareness.
- ◆ Consider retraining or updating seminars in the future to maintain training awareness and currency.
- ◆ Establish contacts in this area of expertise with other DOE sites. Trained personnel from other sites might be called in to supplement or relieve Hanford personnel, and visa-versa.

References

DOE Order 5480.28, *Natural Phenomena Hazard Mitigation*, (01-15-93), U.S. Department of Energy, Washington, D.C.

Wagenblast - 1, G. R., WHC-SD-GN-504, Rev. 0, "Hanford Site Post-NPH Building Inspection Plan," (09-12-96), Westinghouse Hanford Company, Richland, WA.

Wagenblast - 2, G. R., WHC-SD-GN-ER-503, Rev. 0, "Procedure for Post-NPH Inspection of Building," (09-12-96), Westinghouse Hanford Company, Richland, WA.

FEMA 267, "Inspection Guidelines: Evaluation Repair, Modification, and Design of Welded Steel Moment Frame Structures," August 1995.

Appendix A

Class Schedule for REI and DE Training

**Post-NPH Structure Assessment
Rapid Evaluation Inspection
Class Schedule**

Tuesday, March 18, 1997

8:00 - 8:10		(Tom Conrads) Introduction
8:10 - 9:00		(Steve Reidel) History of Past Hanford NPH Events
9:00 - 10:00		(Steve Faulk) Overview of DOE Hanford NPH Response Plan
10:00 - 10:15		(Break)
10:30 - 11:30		(Martin Johnson) Inspector Qualifications Hanford NPH Threshold Events General Earthquake Hazards
11:30 - 12:00		Evaluation Forms Description of Rapid Evaluation Method Hazardous Materials
12:00 - 1:00		(Lunch Break)
1:00 - 1:45		Structure Behavior (Basics) Common Defects/Areas of Damage Evaluation of Damage by Construction Type
1:45 - 2:15		Cast-in-Place Concrete
2:15 - 4:00		Precast Concrete

Wednesday, March 19, 1997

8:00 - 8:30		Q & A Review of Previous Day
8:30 - 9:00		Masonry
9:00 - 9:30		Steel
9:30 - 10:00		Wood Frame and Portables
10:00 - 10:15		(Break)
10:15 - 11:00		Nonstructural, Human Factors
11:00 - 11:30		Discussion
11:30 - 12:00		Multiple Choice Exam

Post-NPH Structure Assessment
Detailed Evaluation
Class Schedule

Friday, March 21, 1997

8:00 - 9:10	Detailed Evaluation Process and Forms
9:10 - 10:00	Evaluation of Damage & In-Situ Strength, (By Construction Type) Concrete, Masonry, Infill
10:00 - 10:15	(Break)
10:15 - 11:15	Steel Moment Frame and Braced Frame
11:15 - 12:00	(Wood and Other)
12:00 - 1:00	(Lunch Break)
1:00 - 1:30	Discussion
1:30 - 2:00	Multiple Choice Exam

Appendix B

Overheads and Handouts for REI Training

Post-NPH Structural Assessments

Presented to:
DOE Hanford Site

Martin W. Johnson

Purpose

- ▶ Implement DOE Order 5480.28
- ▶ "Facilities or sites which have SSCS in PC3 or PC4 shall have procedures to inspect the facility for damage following an NPH event, to place the facility into a safe configuration when damage occurs, and to document and report such damage"

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Objective

- ▶ Establish a team of personnel trained to follow post-NPH inspection procedures
- ▶ **Rapid Evaluation** procedures intended for building inspectors and persons with some significant building design, construction or inspection experience
- ▶ **Detailed Evaluation** procedures intended for persons trained and experienced in structural, civil or mechanical engineering fields and who have a significant degree of building design, construction or inspection experience

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Scope

- ▶ Post-NPH inspection of facilities having nuclear safety, hazardous material safety, life safety, emergency response or programmatic importance at the Hanford site

Not In Scope:

- ▶ Procedures for repair or qualification for restart or operation

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Outline

- ▶ DOE Hanford Response Plan for NPH Events
- ▶ Common Building Construction Types at Hanford
- ▶ Hanford NPH Event Scenarios
- ▶ Two-Stage Evaluation Process Overview
- ▶ Rapid Evaluation Process
- ▶ Detailed Evaluation Process

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DOE Hanford Response Plan For NPH Events

- ▶ Threshold Event
- ▶ Facility Management Actions
- ▶ Roles and Responsibilities
- ▶ Utilization of Inspectors

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DOE Hanford Response Plan Threshold Event

Cataclysmic Events easily trigger emergency response, more moderate events need to pass tests

- ▶ On-site personnel can make initial judgments whether significant damage or unsafe conditions exist
 - Significant damage may cause a moderate event to be classified as a operational emergency
 - Untrained site personnel are not able to placard buildings as "safe" or "unsafe"
- ▶ Pre-established magnitude thresholds, in terms of shaking intensity, recorded wind speed, or ashfall depth can cause an event to become classified as an operational emergency

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DOE Hanford Response Plan Facility Management Actions

- ▶ After a NPH Event is classified as a site emergency, facility management will take the following actions:
 - Mitigate emergencies involving actual or potential hazardous material releases using plant procedures
 - Inspection may be required to allow re-entry for mitigative activities
 - Determine appropriate protective actions for site personnel
 - Recommend appropriate protective actions for the public to responsible agencies by contacting the Patrol Operations Center
 - Activate the *Emergency Operations Center* (EOC) by contacting the Patrol Operations Center
 - Establish an *Incident Command Post* to coordinate emergency activities
- ▶ Upon completion of emergency activities, facility re-entry could be allowed for remaining building safety inspections

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DOE Hanford Response Plan Roles and Responsibilities

- ▶ A Post-NPH Technical Representative will be positioned at the EOC who will be responsible for:
 - Contacting and assembling inspection teams
 - Prioritizing inspections
 - Working with building management to coordinate inspections
 - Tracking completion of inspections and documentation of results
- ▶ Some immediate inspections of critical concerns will occur in the emergency phase (other inspections would likely be during the recovery phase.)
- ▶ Inspectors would reenter only upon approval of the Incident Commander
- ▶ Inspectors assigned to conduct post-NPH evaluations will have received training on NPH hazard evaluation building inspection and approval by the facility manager

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DOE Hanford Response Plan Utilization of Inspectors

Rapid Evaluation Inspector (REI)

- ▶ Initial Rapid Evaluation inspections will be conducted at the discretion of the Incident Commander/Building Emergency Director. Inspections may be required before or in conjunction with any building reentry to ensure that emergency responders, rescuers, etc., are not put in an undue risk situation
- ▶ REIs who are not at work when the NPH event occurs should report directly to their Incident Commander/Building Emergency Director
- ▶ REIs will participate as members of a reentry team to perform building inspections

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DOE Hanford Response Plan Utilization of Inspectors

Building Emergency Director (BED)/Incident Commander (IC)

- ▶ Results of building inspections will be reported to the BED/IC as well as the inspector's availability for reassignment
- ▶ The BED/IC will ensure the results of the building inspection is provided to the EOC Site Emergency Director

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DOE Hanford Response Plan Utilization of Inspectors

Post-NPH Technical Representative (Post-NPH TR)

- ▶ The Post-NPH TR in the EOC will coordinate Rapid Inspections to locations where there is no emergency response organization at the facility. These inspections will occur after any site-wide emergency conditions have been stabilized
- ▶ The Post-NPH TR will assign Detailed Evaluation teams to highest-priority facilities requiring detailed evaluation. Detailed inspections will be prioritized and coordinated through the EOC. If facilities have persons qualified as Detailed Inspectors, they will be requested to provide them to the EOC as a resource as soon as they are available.

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DOE Hanford Response Plan Utilization of Inspectors

Post-NPH Technical Representative (Post-NPH TR)

- ▶ Detailed evaluation teams will perform Rapid Evaluation (if not already completed) and detailed evaluation and report to the Post-NPH TR the results of their inspection and availability for reassignment
- ▶ The Post NPH TR will ensure information regarding detailed and rapid evaluations is communicated to the appropriate authority in the EOC (i.e., site Emergency Director or Onsite Recovery Manager).
- ▶ The need for continued rapid or detailed evaluations will be coordinated by the Post-NPH TR and ensure it is factored into recovery planning

Program of Emergency Response (POEP) NPH TR

Rapid Evaluation Inspector Qualifications

- ▶ Minimum 5 years of experience in general building design, construction or inspection
- ▶ Experience in construction materials of the building being inspected
- ▶ Training to Hanford building inspection procedures
- ▶ Approved by the facility manager

Program of Emergency Response (POEP) NPH TR

Detailed Evaluation Inspector Qualifications

- ▶ Engineers with degrees or equivalent work experience in structural, civil or mechanical engineering fields
- ▶ 5 years combined field and analysis experience in design and construction
- ▶ Experience with construction materials of the building being inspected
- ▶ Training to Hanford building inspection procedures
- ▶ Approved by the facility manager

Program of Emergency Response (POEP) NPH TR

Facility Representative Qualifications

- ▶ Complete knowledge of facility safety rules, restricted areas, building systems, equipment and their safety class functions
- ▶ Must be designated by the facility manager
- ▶ Responsible for obtaining access permission, craft support and removal of in-the-way equipment (i.e. false ceilings, wall covering, etc.)
- ▶ May perform duties of the rapid evaluation and detailed evaluation inspector when qualified

Program of Emergency Response (POEP) NPH TR

Hanford NPH Threshold Events

Hanford NPH Threshold Events

- ▶ Ash Fall - if depth is greater than 5 uncompacted, or 2.5 compacted
- ▶ Flood - if upstream dam failure
- ▶ Wind - greater than 75 mph (PC1, PC2); 80 mph (PC3); or 90 mph (PC4)
- ▶ Earthquake - likelihood of damage varies by performance category

Figure 4-10: NPH Threshold Events

Pre-established Earthquake Thresholds

Conforming To	Effective Ground Acceleration
PC1	0.06 g's
PC2	0.08 g's
PC3	0.13 g's
PC4	0.24 g's

Nuclear facilities equipped with seismic instrumentation have special shut-down thresholds where ground motions exceed the Operating Basis Earthquake (OBE). These thresholds may be either greater or less than the nominal accelerations shown above.

Figure 4-11: Pre-established Earthquake Thresholds

General Earthquake Hazards

- ▶ Surface Rupture (unlikely at Hanford Facilities)
- ▶ Ground Shaking
- ▶ Soil Liquefaction (unlikely at Hanford)
- ▶ Landsliding (unlikely at Hanford Facilities)
- ▶ Subsidence

Figure 4-12: General Earthquake Hazards

Surface Rupture

- ▶ Direct cause and primary hazard of earthquakes
- ▶ Highly destructive to development located along fault
- ▶ Confined to narrow band directly along fault

Figure 4-10a (02) 4/11/11

Ground Shaking

- ▶ Most widespread & damaging effect of earthquakes
- ▶ Intensity varies with:
 - Magnitude
 - Distance to fault
 - Soil type

Figure 4-10b (02) 4/11/11

Measurement of Earthquakes

- ▶ Earthquake Event Magnitude can be measured by several scales
 - Richter Magnitude
 - Coda Length Magnitude (Hanford uses)
- ▶ Intensity of Shaking generally decreases with distance from center (or line) of event
- ▶ Intensity of shaking at any location can also be measured by several scales
 - Modified Mercalli (MMI) scale
 - Maximum horizontal acceleration, as % gravity

Figure 4-11 (02) 4/11/11

Modified Mercalli Intensity Scale

▶ I - Not felt	▶ VII - General Alarm
Chimneys break
Weak buildings damaged
▶ II - Felt in tall bldgs.	▶ VIII - Collapse of poor construction
▶ III - Like passing truck	▶ IX - Partial collapse of ordinary bldgs.
▶ IV - Doors, windows rattle	▶ X - Extensive collapse
▶ V - People wakened	▶ XI/XII - Total destruction
▶ VI - People frightened * Bells ring * Dishes break * Tall objects topple	

Figure 4-12 (02) 4/11/11

Soil Liquefaction (Highly Unlikely at Hanford)

- ▶ Temporary quicksand-like condition
- ▶ Problem in:
 - loose sandy & silty soils
 - high water table
- ▶ Results in extreme settlements

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Lifelines & Infrastructure

- ▶ Electric Power
 - in zones of MMI VII and greater expect service interruptions of 1 to 7 days
- ▶ Natural Gas
 - Gas may be shutoff for several days to a few weeks

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Lifelines & Infrastructure

- ▶ Telephone service:
 - Unreliable for 24 hours
 - Poor service for several days
 - Cellular phones perform better
 - Local exchanges out until power restored

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Lifelines & Infrastructure

- ▶ Major roadways and bridges generally open
 - some detours necessary
 - expect traffic delays

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Lifelines & Infrastructure

► Potable Water

- "Boil water" order in effect for a few weeks
- Limited flows and pressure for several days to a week

Program/Design/POST/0001/0001.ppt

Earthquake Aftershocks

- After a strong earthquake, several moderate to strong aftershocks may occur
 - significant threat to evaluation and search and rescue teams working in heavily damaged structures
 - can "reset" the entire evaluation process by causing re-inspection of previously evaluated buildings
 - posted notices on buildings can become out of date, re-evaluation teams need to check date of notice before entering

Program/Design/POST/0001/0001.ppt

The Two-Level Evaluation Process

The Two-Level Evaluation Process

- Initial Rapid Evaluation of every building on list
 - quickly designate apparently safe and obviously unsafe structures
 - PC1 and PC2 structures which are "safe" (green) or "unsafe" (red) will not receive detailed evaluation
- Detailed Evaluation is intended for all PC3 and PC4 structures plus all "limited entry" (yellow) PC1 and PC2 structures

Program/Design/POST/0001/0001.ppt

Posting Classification System

Posting Classification	Color	Description
Safe	Green	No apparent hazard found, although repairs may be required. Original lateral load capacity not significantly decreased. No restriction on use or occupancy.
Limited Entry	Yellow	Dangerous condition believed to be present. Entry permitted for emergency purposes only. No usage on a continuous basis is allowed. Possible major aftershock hazard.
Unsafe	Red	Extreme hazard, may collapse. Imminent danger of collapse from an aftershock. Unsafe for occupancy or entry. No admittance except by authorized personnel.
Area Unsafe	Red	Area contains serious hazard and is unsafe. No admittance except by authorized personnel.

PostingClassificationSystem.ppt

Posting Classification System

- Unsafe areas
 - Either inside or outside of a building
 - Within striking distance of a falling object or within a danger zone of other hazards
 - Must be corded off or barricaded to prevent entry
- Posting
 - Mount appropriate placard in clearly visible place near all entrances
 - "Do not cross line" tape should be used with the area unsafe placard attached to a perimeter barricade or to the structure in a conspicuous place

PostingClassificationSystem.ppt

Posting Classification System

- Changing of Posting
 - Posting, reposting and removal of tags shall be treated as danger tags in accordance with WHC-CM-1-10, WKS8.2.

PostingClassificationSystem.ppt

Building Posting System

- Green Tag -
 - no apparent structural hazards found
 - occupancy permitted
- Yellow Tag -
 - damage found
 - restricted use
- Red Tag -
 - serious damage found
 - unsafe to occupy

BuildingPostingSystem.ppt

Red Tag

- ▶ Access not permitted
- ▶ Red tag can be placed due to structural or non-structural damage
- ▶ Engineer must evaluate damage & recommend repairs
- ▶ Shoring may be required prior to restoration of limited occupancy
- ▶ Exits may be blocked
- ▶ Repair may or may not be feasible

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Rapid Evaluation Inspection Method

Rapid Evaluation Inspection (REI) Method

- ▶ To quickly inspect buildings and post as apparently safe or obviously unsafe
- ▶ Buildings that do not fall into these two categories are to be posted as limited entry and identified for a more thorough detailed evaluation
- ▶ Re-Entry for inspection shall be in accordance with site Emergency Re-entry Procedure and approved by IoCo

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Rapid Evaluation Inspection (REI) Procedure

- ▶ Examine outside of structure for obvious structural distress (i.e. wall cracks, building settlement)
- ▶ Examine ground in the general area of the structure for fissures, bulged ground or signs of slope movement
- ▶ Evaluate the structure using REI criteria & complete Rapid Evaluation Form

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Rapid Evaluation Inspection (REI) Procedure

- ▶ Inspect building interior for gross non-structural distress
 - fallen ceiling, badly damaged partitions visible from the outside
 - do not enter obviously unsafe structures
- ▶ Post the structure according to results of the evaluation
- ▶ Notify BED/IC of posting
- ▶ BED/IC notifies emergency operation center of posting

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Hazardous Materials

- ▶ Information should be available (obtain information prior to entry)
 - Radioactive material (solids, liquids, or airborne)
 - Asbestos
 - Toxic gas
 - Reactive solvents, acids, or caustics
- ▶ Be alert and cautious; pay attention to sensory clues
- ▶ Do not enter buildings potentially containing radioactive or other hazardous materials without proper monitoring and personal protective equipment
- ▶ For other hazards, document any suspicions, contact the emergency command center for trained personnel to further investigate
- ▶ Post placard accordingly and barricade entry to avoid unnecessary entry
- ▶ Evacuate area immediately and post placards in adjacent structures if possible

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Building Hazards Inspection

- ▶ Building hazards
- ▶ Always check building utilities first
 - Electricity on or off?
 - Water service on or off?
 - Natural gas service on or off?
 - Fire suppression system operational?
 - Telephone communications operational?

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Internal Safety Inspections

- ▶ Internal safety inspections
 - Conduct a sight, smell and hearing survey of each room or area BEFORE entering. Note and avoid any hazardous situations
 - Follow a predetermined route and stay with your partner
 - Be prepared for aftershocks
 - Carry personal supplies with you

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Reports and Documentation

- ▶ Reports and documentation
 - Use the standard REI checklist report format for each specific building
 - Take pictures. If possible use Polaroid instant processing film
 - Use microcassette voice recorders and/or video cassette recorders if available
 - Have a separate file for each building

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Surveying Buildings For Damage

- ▶ Always start on the outside
 - Walk completely around building exterior and note signs of damage
 - Observe roof condition (if accessible from outside)
- ▶ Go inside only if:
 - No RED TAG conditions on outside
 - You are accompanied by some one else (Buddy System)
 - Someone knows you are going in and is available to get help

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Building Damage: Structural vs. Non-structural

▶ Structural Elements - Hold up the building ▶ beams ▶ columns ▶ braces ▶ floors ▶ exterior walls	▶ Non-structural Elements - Make the building attractive and usable ▶ partitions ▶ lighting fixtures ▶ HVAC equipment ▶ furnishings ▶ equipment

Both Can Present Hazards!

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Structural Damage

- ▶ Can lead to building collapse
- ▶ Repair may requires costly demolition and replacement of finishes
- ▶ A structural engineer is always required

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Non-structural Damage

- ▶ Will not lead to building collapse but can pose hazards
- ▶ Can be repaired with sketches prepared by a structural engineer and approved by site chief structural engineer - (complete construction documents may not be required)

Figure 4.10a (POET 10/11) .jpe

Common Structural Defects Often Lead to Damage

- ▶ Inadequate strength
- ▶ Inadequate Ductility
- ▶ Building Codes regulate the minimum strength and minimum ductility provided in buildings
- ▶ Pre-Mid 1970s buildings are generally more subject to damage

Figure 4.10b (POET 10/11) .jpe

Common Structural Defects Vertical Irregularity - Soft Stories

- ▶ Buildings with weak or "soft" first stories tend to be subject to greater damage
- ▶ Look for concentrations of damage in the lower story
- ▶ Offset in vertical configurations and lateral systems
- ▶ Look for damage concentrated at the building interfaces

Figure 4.10c (POET 10/11) .jpe

Common Structural Defects Plan Irregularity

- ▶ Irregular building geometries create stress concentrations
- ▶ Look for damage at:
 - re-entrant corners or building offsets
 - large openings in floors, roofs

Figure 4.10d (POET 10/11) .jpe

Common Structural Defects "Pounding"

- Occurs when buildings are located too close together
- Look for damage to both buildings at lowest adjacent point of contact
- More critical if roof/floor of one building "pounds" at mid-column level of adjacent building

Figure 4.10a (HNF-SD-GN-ER-505) 10/01/01

Common Structural Defects Heavy Cladding

- Exterior walls of concrete or masonry can become dislodged from structure
- Look for wall panels that are leaning outward

Figure 4.10b (HNF-SD-GN-ER-505) 10/01/01

Assessing Damage Due to Vertical Overloads (Such as Ash Fall)

- Well-designed beams and slabs will tend to develop noticeable sag before failure
 - Concrete construction will show large number of thin cracks (< 1/8 in.)
 - Wood beams/joists will emit audible splitting or cracking sounds before breaking
 - Sagging elements are unable to support additional load unless well anchored at ends (catenary effect)
 - Sagging elements can experience increasing load due to ponding effect
- Poorly designed beams and slabs may fail with little warning
 - Concrete elements will show a few wide cracks or splits (1/2 in. wide or more)
 - Steel elements may have broken bolts, signs of tearing at connections, or signs of rolling over onto side

Figure 4.11 (HNF-SD-GN-ER-505) 10/01/01

Hanford Building Construction Types

Hanford Building Construction Types

- ▶ Cast-in-place concrete shear wall "boxes" & moment-resisting frames
- ▶ Pre-cast concrete including tilt-up
- ▶ Reinforced & unreinforced masonry
- ▶ Steel moment-resisting frames & braced frames
- ▶ Industrial light metal frame
- ▶ Portables
- ▶ Wood

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Damage Review for 10 Common Types of Buildings

- ▶ High Risk Buildings
 - Unreinforced Masonry Bearing Wall
 - Reinforced Concrete Frame
 - Tiltup or Precast Wall
 - Frame with Infill Masonry
 - Precast Concrete Frame
- ▶ Moderate Risk Buildings
 - Wood Frame
 - Steel Frame
 - Reinforced Concrete Shear Wall
 - Reinforced Masonry
 - Light Metal Buildings

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Reinforced Concrete Frame Buildings (RCFs)

- ▶ Common in mid rise construction; 4 - 15 stories
- ▶ Constructed from 1920 to present
- ▶ Many pre-1973 structures hazardous due to non-ductile detailing

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Identifying RCF Buildings

- ▶ Beams & Columns result in grid-like exterior appearance
- ▶ Exposed concrete columns and beams
- ▶ Windows at least as wide as columns (often wider)

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Common Damage to RCFs RED TAG Conditions

- ▶ Cracking in excess of 1/4" width
- ▶ Excessive spalling of concrete
- ▶ Exposed reinforcing steel
- ▶ Concrete core integrity compromised

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Common Damage to RCFs YELLOW TAG Conditions

- ▶ Cracking in beams, columns or joints - with widths less than 1/4"
- ▶ Minor spalling
- ▶ Loose concrete

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Infill Frame Buildings (IFBs)

- ▶ Typically mid to high-rise construction (5 - 30 stories)
- ▶ Constructed circa 1900-1940
- ▶ Commercial, institutional, and residential occupancies

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Identifying IFBs

- ▶ Perimeter walls of brick or stone masonry
- ▶ Interior walls of hollow clay tile
- ▶ Ornate terra cotta trim on exterior
- ▶ Many window openings
- ▶ Thick walls

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Common Damage IFB Buildings RED TAG Conditions

- ▶ Large width ($>1/4$ " cracking of exterior masonry)
- ▶ Toppled parapets
- ▶ Fallen veneers or ornaments
- ▶ Spalled hollow clay tile partitions
- ▶ Spalled concrete or exposed reinforcing

Figure 4-10a (P002) 4/11/11 jw

Common Damage IFB Buildings YELLOW TAG Conditions

- ▶ Minor cracking of exterior or interior masonry ($<1/4$ inch width)
- ▶ Loosened trim and ornamentation

Figure 4-10b (P002) 4/11/11 jw

Concrete Tiltup Buildings or Precast Wall

- ▶ Common post-1960 construction
- ▶ Low rise - industrial and commercial occupancies
- ▶ Pre-1976 buildings may be hazardous
- ▶ "Lessons" from Northridge Earthquake

Figure 4-11a (P002) 4/11/11 jw

Identifying TU Buildings

- ▶ One or two story
- ▶ Exterior walls of concrete
- ▶ Vertical joints in walls every 16 to 24 feet
- ▶ Relatively flat or barrel shaped roof
- ▶ Wood framed (or metal deck) roof

Figure 4-11b (P002) 4/11/11 jw

Precast Wall Buildings

- ▶ One or two story
- ▶ Walls & roof concrete
- ▶ Vertical joints every 6 to 10 feet
- ▶ Relatively flat roof
- ▶ Walls may be flat, tee, or double tee section
- ▶ Roof/floors may be flat (span crete) tees or double tee sections

Figure 4-10a (PCF) 4/10/10

Common TU Damage RED TAG Conditions

- ▶ Wall panels leaning out from building
- ▶ Partial roof collapse
- ▶ Cracking in wall panels exceeding 1/4"
- ▶ Exposed reinforcing in wall panels
- ▶ Spalling or large cracking at beam seats on walls/columns

Figure 4-10b (PCF) 4/10/10

Common TU Damage YELLOW TAG Conditions

- ▶ Cracks in roof membrane
- ▶ Cracks in wall panels, narrower than 1/4"
- ▶ Cracks in beam seats at walls

Figure 4-10c (PCF) 4/10/10

Precast Concrete Frame (PCF) Structures

- ▶ Commonly used for parking garages, hotels, mid-rise office buildings
- ▶ Typically post-1960
- ▶ Beams, columns, walls cast separately and then connected together in field

Figure 4-10d (PCF) 4/10/10

Identifying PCF Buildings

- ▶ Exposed concrete beams and columns in grid patterns
- ▶ Beams and columns look "put-together"
- ▶ Large window areas all sides

Figure 4-10a (PCF) 4/11/11 pgs

Common PCF Damage RED TAG Conditions

- ▶ Precast elements dislodged or leaning away from building
- ▶ Large width cracking of concrete ($> 1/4"$)
- ▶ Spalled concrete
- ▶ Exposed reinforcing steel
- ▶ Building leans to side

Figure 4-10b (PCF) 4/11/11 pgs

Common PCF Damage YELLOW TAG Conditions

- ▶ Minor cracking of concrete beams, columns, floors ($< 1/4"$ width)
- ▶ Precast connections exhibit minor movements ($< 1/2"$)

Figure 4-10c (PCF) 4/11/11 pgs

Unreinforced Masonry Bearing Wall Buildings (URMs)

- ▶ Clay brick, stone, adobe, or concrete block walls - hold up building
- ▶ Walls lack reinforcing steel bars
- ▶ Most have wood roofs and floors
- ▶ Most built prior to 1940

Figure 4-10d (URM) 4/11/11 pgs

Identifying URM Buildings

- ▶ Typically 4 stories or less in height
- ▶ Usually has visible masonry (brick, stone, etc.) on exterior
- ▶ May have anchor plates (rosettes) on walls at floor levels
- ▶ Small windows
- ▶ Arched lintels

Image of URM Building 001 001.jpg

Common Damage to URM RED TAG Conditions

- ▶ Loss of parapets
- ▶ Loss of veneer masonry
- ▶ Diagonal cracking of walls with offset
- ▶ Walls lean out from building
- ▶ Walls collapse
- ▶ Neighbor building has above conditions

Image of URM Building 002 001.jpg

Common Damage to URM YELLOW TAG Conditions

- ▶ Horizontal cracking across base of parapets
- ▶ Horizontal cracking across base of piers
- ▶ Diagonal cracking of piers (less than 1/4" width) without offset
- ▶ Extensive plaster damage interior walls

Image of URM Building 003 001.jpg

Reinforced Concrete (RCSW) and Masonry (RMSW) Shear Wall Buildings

- ▶ Low and mid-rise construction
- ▶ Typically constructed post-1930
- ▶ Characterized by relatively solid walls
- ▶ Commercial, institutional and residential uses

Image of RCSW and RMSW Shear Wall Building 001.jpg

Identifying RCSWs and RMSWs

- ▶ Window and door openings usually smaller than solid portions of walls
- ▶ Typically exterior walls of concrete or concrete block construction
- ▶ Usually concrete slab or wood floors

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Common Damage RCSW & RMSW Buildings

▶ RED TAG DAMAGE - Extensive cracking > 1/4" width - Spalled concrete or masonry - Exposed reinforcing steel	▶ YELLOW TAG DAMAGE - Widespread but moderate cracking (crack widths less than 1/4")

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Steel Frame (SF) Buildings

- ▶ Low-, Mid- and High-rise construction
- ▶ Typically post 1950
- ▶ Commercial, industrial and institutional uses
- ▶ Steel beams and columns with concrete floors, sometimes diagonally braced

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Identifying SF Buildings

- ▶ Beams and columns give a grid-like appearance on all sides
- ▶ Many very large windows
- ▶ Frequently have glass exteriors
- ▶ Underside of floors commonly metal deck

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Common SF Building Damage

- ▶ Extensive damage to architectural elements - ceilings, windows, piping
- ▶ Some permanent out-of-plumb condition may be noticeable

Structural damage to Steel Frame Buildings is typically difficult to detect. If architectural damage occurs - the building should be evaluated by a structural engineer.

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Light Metal Buildings (LMBs)

- ▶ Constructed of steel beams and columns with metal siding
- ▶ Typically 1 story
- ▶ Large open interior spaces
- ▶ Agricultural, industrial and commercial uses

Page 1 of 10

Identifying LMs

- ▶ Metal Siding and Roofs
- ▶ Steel columns and beams visible inside
- ▶ Diagonal rod braces adjacent to walls

Page 1 of 10

Common LM Building Damage

▶ RED TAG

- Building noticeably out of plumb
- Diagonal rod braces broken or torn loose from connections
- Damage to moment connections

▶ YELLOW TAG

- Stretched or buckled rod braces
- Building siding is separating from structure

Page 1 of 10

Wood Frame Buildings (WFBs)

- ▶ Typically 3 stories or less
- ▶ Typically less than 20,000 sq ft. floor area per floor
- ▶ Walls constructed of plywood sheathing or stucco, gypsum board, or plaster over wood studs

Image of a wood frame building

Identifying WFs

- ▶ Most low-rise residential construction
- ▶ Many low-rise commercial construction
- ▶ Exterior walls of stucco, wood siding, or scored plywood

Image of a wood frame building

Common WF Damage RED TAG Conditions

- ▶ Building slid off foundation
- ▶ Story noticeably out of plumb
- ▶ Extensive damage to plaster stucco and gypsum board walls and ceilings

Image of a wood frame building

Common WF Damage YELLOW TAG Conditions

- ▶ Moderate cracking of finishes
- ▶ Doors and windows jammed
- ▶ Ornamentation (signs, chimneys, etc.) loose

Image of a wood frame building

Human Factors

Personal Protection

- ▶ Don't be a victim
 - Respiratory protection
 - Eye protection
 - Head and shoulder protection
 - Hand and finger protection
 - Foot and ankle protection
 - Personal identification
 - Work in teams

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Personal Items

- Don't be a victim*
- ▶ Suggested Items:
 - Canteen & food
 - Water purification tablets
 - Spare clothing
 - Safety vests
 - Gloves
 - Knee pads
 - Sleeping bag
 - First aid kit
 - Insect repellent
 - Binoculars
 - ▶ Essential Items:
 - Flashlight & extra batteries
 - Spare clothing (including passport/visa when appropriate)
 - Official identification
 - Driver's license
 - Credit card/cash/traveler's checks (including change for pay phones)
 - Eye glasses/safety glasses (including extras)
 - Prescription medication
 - Hard hat
 - Socks/sundry shoes
 - Spare clothing
 - Personal hygiene supplies
 - Dust mask or respirator
 - Gloves
 - Gasoline (in vehicle)

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Field Equipment

- ▶ Suggested Items:
 - Battery operated lanterns and extra batteries
 - High capacity cooler, and copy materials
 - Flashlight and extra batteries
 - Compass & level
 - Calculator
 - Portable battery-powered radio
 - First aid kit
 - Binoculars
 - Map recorder, blank tapes and batteries
 - Tape measure
 - Magnetic compass
 - Weatherproof notebook
- ▶ Essential Items:
 - Official identification/field passes
 - Inspection forms
 - Posting placards
 - Maps and maps of the area
 - State and/or federal standards
 - Communications equipment (walkie-talkies, etc.)
 - Paper, pens, staples, etc.
 - Names and phone numbers of emergency personnel
 - Paper notebook
 - Pen/pencil
 - Flashlight and extra batteries
 - Compass and level
 - Equipment, and extra batteries

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Human Factors Following An Earthquake (And Other Disasters)

- ▶ Human behavior varies w/ disaster and extent of damage (both personal and community)
- ▶ Dealing with occupants and building owners of damaged property
- ▶ Coping with stress in the field
- ▶ Providing support to inspectors in the field

Figure 4-10 HNF-SD-GN-ER-505

Human Behavior

- Varying degrees of fear
- Humans are very adaptive and regain composure in a short period of time and often proportional to response of governing agencies and aid organizations
- Disaster victims are normal people subjected to severe stress and strain
- Personal concerns. The greater the personal exposure the greater the distress
- "The Disaster Syndrome" - individuals remain dazed or stunned
- "Counter-Disaster Syndrome" - individuals deny fear, anxiety, and grief
- "Long Term Effects" - feelings of overwhelming stress, grief, anger and inadequacy. Symptoms include insomnia, digestive upsets, depression, and nervousness

Figure 4-11 HNF-SD-GN-ER-505

Dealing With Occupants And Building Owners

- Make an effort to be helpful and understanding. You may be the first person or authority encountered (analogous to a physician, people want "the personal touch"). This should also expedite the inspection
- Balance people's needs with the process. Be prepared to factually answer questions about the earthquake. Remember to be prepared to direct people to the building authority or disaster aid centers
- Difficult and suspicious owners should be dealt with very professionally. Be objective, factual and patient
- Buildings declared "unsafe" - be sure to explain the hazards and risks noted on the placard to the owner
- Trade with partner coping with the owner or occupants to avoid conflict
- The inspector's task also entails encountering victims and treating them w/ understanding, offering reassurance and factual information, and directing them to further aid

Figure 4-12 HNF-SD-GN-ER-505

Coping With Stress in the Field

- Long hours of physical exertion, emotional turbulence, unfamiliar environments, loss, destruction, injury and possibly death, lack of sleep or food, separation from one's family and the subjugation of one's own needs over the concerns of others can impact the emotional and physical well-being of inspectors
- "Burn-out Syndrome" - State of exhaustion, irritability, and fatigue (which creeps up unrecognized and undetected) decreases the effectiveness and capability of the person
- Specific Symptoms and Warning Signs:
 - 1) Thinking Symptoms: Inability to make decisions, judgements, slowness of thought and mental confusion inability to express one's self orally or verbally
 - 2) Psychological Manifestations: Depression, irritability, anxiety and hyper excitability
 - 3) Physical Warnings: Exhaustion, loss of energy gastrointestinal distress, appetite disturbances and sleep disorders

Figure 4-13 HNF-SD-GN-ER-505

Stress Relieving Measures

- Develop a "buddy" relationship with a co-worker to monitor the personal effects of the disaster
- Encourage and support co-workers
- Talk about your experiences, feelings and fears and listen to others around you
- Try to exercise and enjoy recreational activities away from the disaster scene
- Try to eat well and get enough sleep
- Pace yourself (i.e. 12 hrs on, 12 hrs off)
- Take a break if effectiveness is diminishing

"You very important to the overall restoration and recovery process of your community"

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Providing Support to Inspectors in the Field

Providing Support to Inspectors in the Field

- Organizational actions and preparation reduce confusion and stress. Be prepared and practice emergency procedures before a disaster
- Be clear with inspection teams about their tasks. Setting priorities and strategies with available resources is an effective use of time
- Define the assessment type and the information required with average time perspectives per square footage
- Designate teams to specific geographical areas to prevent redundant inspections
- Provide an inside emergency number where inspectors only call for reporting and requesting special assistance
- Interim meetings among inspection teams to provide policy updates and agency support of teams. Encourage social interaction after hours for stress management
- Publicizing with the local media is best handled by public relations professionals
- Carry visible and distinct identification with you at all times

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2.0 SAFETY EVALUATION PROCEDURE OUTLINE

Performing a safety inspection of a building immediately following a natural force event is accomplished using a two level procedure. This procedure calls for an initial Rapid Evaluation of every building. This is normally the first level of evaluation and is designed to quickly designate the apparently safe and obviously unsafe structures. PC1 and PC2 buildings that are concluded to be apparently safe require no further inspection and may be occupied. PC1 and PC2 buildings that do not fall into either of the two preceding categories are designated as having limited entry, and along with all PC3 and PC4 buildings, regardless of their initial posting, require a more detailed inspection by a structural engineer. This inspection, designated the Detailed Evaluation, is the second level of inspection and is designed to result with a rating of structures as either safe or unsafe. The safety inspection procedure for PC1 and PC2 buildings is shown diagrammatically below.

SAFE

NO RESTRICTION ON USE OR OCCUPANCY

This structure has been inspected (as indicated below) and no apparent structural hazard has been found. Report any unsafe conditions to Emergency Control Center; reinspection may be necessary.

Date _____

Time _____

Exterior Only

☐

Exterior and Interior

☐

Building Name and Location:

Inspector(s) Name/Signature

DO NOT REMOVE THIS PLACARD UNLESS
AUTHORIZED BY THE EMERGENCY CONTROL CENTER

LIMITED ENTRY

OFF LIMITS TO UNAUTHORIZED PERSONNEL

Warning:

This structure has been damaged and its safety is questionable. Entry is not allowed except by authorized personnel. Seismic aftershocks or other natural phenomenon events may result in building collapse causing death or injury.

Date _____

Time _____

Enter For Emergencies Only

Other

☐ _____☐ _____

Building Name and Location:

Inspector(s) Name/Signature

DO NOT REMOVE THIS PLACARD UNLESS
AUTHORIZED BY THE EMERGENCY CONTROL CENTER

UNSAFE

DO NOT ENTER OR OCCUPY

Warning:

This structure has been seriously damaged and is unsafe. Do not enter. Entry may result in death or injury.

Date _____

Time _____

Comments:

Building Name and Location:

Inspector(s) Name/Signature

DO NOT REMOVE THIS PLACARD UNLESS
AUTHORIZED BY THE EMERGENCY CONTROL CENTER

AREA UNSAFE

DO NOT ENTER OR OCCUPY

Warning:

This area contains a serious hazard and is unsafe. Do not enter. Entry may result in death or injury.

Date _____

Time _____

Description of Hazard:

Building Name and Location:

Inspector(s) Name/Signature

DO NOT REMOVE THIS PLACARD UNLESS
AUTHORIZED BY THE EMERGENCY CONTROL CENTER

Table 4.1 Basic Rapid Evaluation Criteria

<u>Condition</u>	<u>Posting</u>
1. Building has collapsed, partially collapsed, or has shifted off its foundation.	UNSAFE
2. Building or any story is significantly out of plumb.	UNSAFE
3. Obvious severe damage to primary structural members, severe racking of walls, or other signs of severe distress present.	UNSAFE
4. Obvious overhead falling hazard present.	AREA UNSAFE
5. Fissures in ground, ground movement, or slope displacement present.	UNSAFE
6. Other obvious hazard present (e.g., fallen power line, toxic spill, spread of radiological contamination or release).	UNSAFE or AREA UNSAFE

Rapid Evaluation Safety Assessment Form

Building Description:

INSPECTION DATE/TIME:

Name: _____

Location: _____

OVERALL RATING:

Inspector(s): Name / Signature

SAFE

☐
 _____ Exterior Only
 _____ Exterior and
 _____ Interior

LIMITED ENTRY

UNSAFE

☐☐☐

Instructions:

Review structure for the conditions listed below. A "yes" answer to 1, 2, 3, or 5 is grounds for posting the entire structure UNSAFE. If more review is needed, post LIMITED ENTRY. A "yes" answer to 4 requires posting AREA UNSAFE and/or barricading around the hazard. Hazards such as radiological release/contamination or toxic spill are covered by 6 and are to be posted and or barricaded to indicate AREA UNSAFE.

Condition: _____

Yes

No

More
Review
Needed

1. Collapse, partial collapse, or building off foundation
2. Building or story noticeably leaning
3. Severe racking of walls, obvious severe damage and distress
4. Obvious overhead falling hazard present
5. Severe ground or slope movement
6. Other obvious hazard present

☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐☐

Recommendations:

- ☐ No further action required
☐ Detailed Evaluation required
☐ Other

Additional Comments:

6-2

CES FORM ITEM	BUILDING ELEMENT	TYPE OF DAMAGE		
		RED (5-6)	GOLD (3-4)	GREEN (1-2)
E1	Exterior Walls	<ul style="list-style-type: none"> • Significantly out of plumb • Severely cracked • Falling hazard • Walls pulled away from horizontal framing at floors and roof • (Refer to G1) 	<ul style="list-style-type: none"> • Minor cracks • Moderate racking • No apparent instability hazard • Localized damage • (Refer to G1) 	<ul style="list-style-type: none"> • Minor cracks • Minor racking • Not a falling hazard • (Refer to G1)
E2 E3 E4	Frame Members & Connections	<ul style="list-style-type: none"> • Signs of yielding • Indication that tension-only bracing members have yielded or gone into compression • Significant cracking throughout • Buckling throughout structure 	<ul style="list-style-type: none"> • Minor distress in members such that it appears frame and connections would withstand aftershock • No apparent instability hazard 	<ul style="list-style-type: none"> • Very minor distress
E5	Roof Framing Roof-Wall Connection	<ul style="list-style-type: none"> • Partial collapse with reason to suspect further collapse in aftershock • Displacement of roof with respect to wall • Significant damage to roof-wall connection • Significant damage to ledger 	<ul style="list-style-type: none"> • No apparent instability hazard • Minor or very localized damage with no reason to suspect significant additional damage in aftershock 	<ul style="list-style-type: none"> • Very minor damage • No apparent damage at roof-wall connection

2.2 Draft SEAONC Disaster Emergency Services Committee building assessment criteria.

CES FORM ITEM	BUILDING ELEMENT	TYPE OF DAMAGE		
		RED (5-6)	GOLD (3-4)	GREEN (1-2)
E6	Interior Bearing Wall/Shear Wall	<ul style="list-style-type: none"> Significantly out of plumb Significant cracking Distress between wall and connections to other building elements 	<ul style="list-style-type: none"> No apparent instability hazard Minor damage at walls or connections between walls and connections to other building elements 	<ul style="list-style-type: none"> Very minor damage Minor cracking
E7	Partitions/Non-Bearing Walls	<ul style="list-style-type: none"> Partial collapse with possibility of further damage in aftershock Falling hazards throughout 	<ul style="list-style-type: none"> No apparent instability hazard Moderate damage but, apparently able to withstand aftershock Very localized or minimal falling hazard 	<ul style="list-style-type: none"> Minor damage
E8	Floors/Floor-to-Wall Connections	<ul style="list-style-type: none"> Partial collapse Lateral or vertical displacement of floor with respect to wall Significant damage or separation at floor-wall connection Significant damage to ledger Significant sponginess in floor above grade Significant cracking in concrete floors 	<ul style="list-style-type: none"> No apparent instability hazard Minor, local sponginess in floor 	<ul style="list-style-type: none"> Minor damage
E9	Stairs	<ul style="list-style-type: none"> Most or all means of egress unsafe or blocked Significant damage at stair connection Partial collapse Debris on stairs 	<ul style="list-style-type: none"> No apparent instability hazard Moderate damage at stair, but not a falling hazard 	<ul style="list-style-type: none"> Minor damage All means of egress safe

DES FORM ITEM	BUILDING ELEMENT	TYPE OF DAMAGE		
		RED (5-6)	GOLD (3-4)	GREEN (1-2)
E10	Elevator	<ul style="list-style-type: none"> Most or all means of egress unsafe or blocked 	<ul style="list-style-type: none"> Elevator or shaft damaged but other means of egress safe 	<ul style="list-style-type: none"> Minor damage to shaft or elevator, but all other means of egress safe
E11	Glass	<ul style="list-style-type: none"> Probability of glass breaking in aftershock to threaten human safety in or around building A large amount of glass on building with a significant amount broken 	<ul style="list-style-type: none"> Minor amount of broken glass Relatively small amount of glass in building at exits 	<ul style="list-style-type: none"> Minor amount of broken glass
E12	Mechanical/ Electrical Equipment Support	<ul style="list-style-type: none"> Unsupported equipment or failed supports with equipment being falling hazard Safety hazards due to broken ducts, piping, downed wires etc. damaged by failed equipment supports 	<ul style="list-style-type: none"> Significant distress to equipment supports, with equipment likely to fall in aftershock Localized damage 	<ul style="list-style-type: none"> Minor distress at equipment supports but with no fire, or life safety hazard
E13				
E14	Porches/ Canopies/Add-on Rooms	<ul style="list-style-type: none"> Significant distress at connections Partial collapse Falling hazards that severely limit safe egress from the building 	<ul style="list-style-type: none"> No apparent instability hazard Moderate localized damage, but appears adequate to withstand aftershock Other means of egress safe 	<ul style="list-style-type: none"> Minor damage No falling or collapse or egress hazard

Figure 2.2 (Continued)

CES FORM ITEM	BUILDING ELEMENT	TYPE OF DAMAGE		
		RED (5-6)	GOLD (3-4)	GREEN (1-2)
E14A	Columns	<ul style="list-style-type: none"> • Significantly out of plumb • Moderate buckling • Severe cracking • Signs of yielding • Falling hazard • Damaged, so as to no longer be providing support to level above 	<ul style="list-style-type: none"> • No apparent instability hazard • Minor cracking 	
E14B	Foundations	<ul style="list-style-type: none"> • Moderate foundation damage • Severe building settlement • Building has slid off its foundation and superstructure is in jeopardy 	<ul style="list-style-type: none"> • No apparent instability hazard • Moderate foundation cracking for buildings on grade • Moderate building settlement 	<ul style="list-style-type: none"> • Minor building settlement • Minor foundation cracking • Evidence of local uplift
G1 G2 G3 G6	Parapet Walls Ornamentation Chimney Roof Equip. or Suspended Equipment	<ul style="list-style-type: none"> • Partial collapse • Moderate cracking • Failed bracing 	<ul style="list-style-type: none"> • No apparent instability hazard • Minor distress at parapet 	<ul style="list-style-type: none"> • No apparent damage
G7A	Storage Shelving	<ul style="list-style-type: none"> • Heavy storage (books, warehouse) unsupported or partially collapsed • Lateral supports showing severe distress 	<ul style="list-style-type: none"> • No apparent instability hazard 	<ul style="list-style-type: none"> • No apparent threat to human safety

CES FORM ITEM	BUILDING ELEMENT	TYPE OF DAMAGE		
		RED (5-6)	GOLD (3-4)	GREEN (1-2)
G7B	Suspended Ceiling/ Light Fixtures	<ul style="list-style-type: none"> Partially collapsed Improperly anchored light fixtures Unbraced ceiling 	<ul style="list-style-type: none"> No apparent instability hazard 	<ul style="list-style-type: none"> No significant damage No apparent threat to human safety
G7C	Adjacent Bldgs.	<ul style="list-style-type: none"> Adjacent buildings are collapse hazards which could impact the building being inspected 	<ul style="list-style-type: none"> No apparent instability in adjacent buildings 	<ul style="list-style-type: none"> No apparent instability in adjacent buildings
H1 H2	Settlement or Liquefaction	<ul style="list-style-type: none"> Significant settlement or liquefaction 		
H3	Landslide	<ul style="list-style-type: none"> Possibility of landslide in aftershock which would threaten human safety or structural integrity Elevated building with significant foundation damage Unstable rock (boulders) formation which could slide or roll into building 	<ul style="list-style-type: none"> Minor, localized slope failures in area where similar occurrence would not pose threat to human safety or to buildings structural integrity 	<ul style="list-style-type: none"> Site located on predominantly flat area with no apparent landslide hazard.
I	Other-Buildings Type of Use	<ul style="list-style-type: none"> Chemical labs, manufacturing buildings, etc. with potential to create fires or hazardous fumes. Fuel/flammable materials plants or any building where chemical or fuel odors can be smelled. 		

Figure 2.2 (Continued)

Appendix C

Overheads and Handouts for Supplementary DE Training

Detailed Evaluation Process

- ▶ What the detailed evaluator should expect
 - Initial assignments may be Rapid Evaluation of most critical structures
 - Quickly become encumbered with follow-up assessment of yellow-tagged "limited-entry" structures
 - Need to prioritize between conducting additional rapid (first pass) evaluations and conducting detailed (second pass) assessments
 - After a significant earthquake strong aftershocks should be expected which may reset the inspection program
 - Shore first, evaluate later

Program/Slide/0001-0001.ppt

Detailed Evaluation Process

- ▶ Evaluating strength and stiffness of damaged conditions
 - California OES Guidelines
 - EERI Spectra Paper

Program/Slide/0001-0001.ppt

Detailed Evaluation Method

- ▶ Establishes whether the building is safe to re-enter and re-occupy and post the structure accordingly
- ▶ Evaluation should be carried out by at least 2 engineers with experience in seismic design of buildings
- ▶ Elevators, safety inspection and qualifications should be treated as suspect and tagged unsafe
- ▶ Do not reset elevators if the protective systems are tripped - leave for a qualified elevator technician

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Detailed Evaluation Inspection Guidelines

- ▶ Examine from the ground the entire outside of the structure
- ▶ Do not enter the building or on the roof
- ▶ Identify & inspect vertical discontinuities where there is a sudden change in stiffness such as a setback or a soft story
- ▶ Identify & inspect irregular configurations of the building in plan as is the case for "L" shape, "T" shape or other complex shaped buildings

Program/Slide/0001-0001.ppt

Detailed Evaluation Inspection Guidelines

- ▶ Look for racking of exterior walls, glass frames, etc. which are symptoms of excessive drift
- ▶ Examine non-structural elements such as curtain walls, parapets, signs, etc. to determine if they could be a potential falling hazard
- ▶ Look for fractures in foundation or exposed lower walls
- ▶ Examine the site for geotechnical hazards noting fissures, bulged ground, etc.

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Detailed Evaluation Inspection Guidelines

- ▶ Inspect the structure from inside the building for structural deficiencies
- ▶ Begin at bottom level and proceed up
- ▶ Inspect stairwells, basements, mechanical rooms and other areas where the structural system is likely to be exposed
- ▶ Inspect the vertical load carrying system for signs of column failure where floor or roof framing has begun to pull away from its vertical supports or where the slab or beam has failed

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Detailed Evaluation Inspection Guidelines

- ▶ Inspect exposed components of the foundation system and the basement floors for fractured components and uneven settlement
- ▶ Inspect for non-structural interior hazards
- ▶ spills or leaks of hazard materials
- ▶ cladding connections
- ▶ obstructed exitways
- ▶ Inspect for hazards associated with mechanical systems, components and equipment

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Detailed Evaluation Inspection Guidelines

- ▶ Evaluate structure using given criteria and complete the Detailed Evaluation Form
- ▶ Post the structure according to the results of the evaluation
- ▶ Notify the IC/BED or Facilities Management and the Emergency Operations Center of the posting

*See
Form*

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Strengthening Options for URM Buildings

- High Benefit
 - Brace parapets
 - Anchor walls to floors/roof
 - Add plywood to floors
 - Add concrete shear walls
- Moderate Benefit
 - Add interior partitions
 - Add braced steel frames
 - Provide posts under beam bearings in walls

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Strengthening Options for RCF Buildings

- High Benefit
 - Base isolation
 - Add concrete shear walls
 - Strengthen concrete frames
- Moderate Benefit
 - Add braced steel frames

Program/Project: HNF-505

Strengthening Tiltup Buildings

- High Benefit
 - Add anchors to wall-roof/ wall-floor connection
 - Add continuous ties to wood roof or floor framing
 - Add interior shear walls or braced frames
 - Add plywood to roof
 - Improve connections between precast elements

Program/Project: HNF-505

Strengthening Options for IFBs

- High Benefit
 - Base isolation
 - Add concrete shear walls
 - Remove hollow clay tile walls
 - Brace parapets
- Moderate Benefit
 - Add braced steel frames

Program/Project: HNF-505

Strengthening Options for PCFs

- ▶ High Benefit
 - Strengthen connections
 - Add shear walls
- ▶ Moderate benefit
 - Add braced steel frames

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Strengthening Options for WFs

- ▶ High Benefit
 - Add foundation bolts
 - Sheath cripple walls with plywood
 - Add plywood sheathing to walls and partitions
- ▶ Moderate Benefit
 - Add frames in soft stories

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Table 5.1 General Detailed Evaluation Criteria

<u>Structural Condition - Description</u>		<u>Postina</u>
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Building or Individual Story Noticeably Leaning.....	UNSAFE
	Fractured Foundations.....	UNSAFE
2. Vertical Load System	Columns Noticeably out of Plumb.....	UNSAFE
	Buckled or Failed Columns.....	UNSAFE
	Roof or Floor Framing Separation from Walls or Other Vertical Supports.....	UNSAFE
	Bearing Wall, Pilaster, or Corbel Cracking Which Jeopardizes Vertical Support.....	UNSAFE
	Other Failure or Incipient Failure of Significant Vertical Load-Carrying Element or Connection.....	UNSAFE
3. Lateral Load System	Broken, Leaning, or Seriously Degraded Moment Frames.....	UNSAFE
	Severely Cracked Shear Walls. $> \frac{1}{4}$ inch	UNSAFE
	Broken or Buckled Vertical Braces.....	UNSAFE
	Broken or Seriously Damaged Diaphragms or Horizontal Bracing.....	UNSAFE
	Other Failure or Incipient Failure of Significant Lateral Load-Carrying Element or Connection.....	UNSAFE
4. P-Delta Effects	Multistory Frame Building with Residual Drift.....	UNSAFE
5. Fallings Hazards	Falling Hazards Present.....	AREA UNSAFE
6. Slope or Foundation Distress	Base of Building Pulled Apart or Differentially Settled, with Fractured Foundations, Walls, Floors, or Roofs...	UNSAFE
	Building in Zone of Faulting or Suspected Major Slope Movement.....	UNSAFE
7. Other Hazards	Spill of Unknown or Dangerous Materials	UNSAFE
	Spread or Release of Radiological Contamination.....	AREA UNSAFE
	Downed Power Lines.....	AREA UNSAFE

☐

This image shows a single sheet of white paper with horizontal ruling lines. The lines are evenly spaced and run across the width of the page. There are no margins, text, or other markings on the paper.

Detailed Evaluation Safety Assessment Form - Checklist

Instructions:

Examine the structure to determine if any hazardous conditions exist. A "yes" answer in categories 1, 2, or 4 is grounds for posting building UNSAFE. A "yes" answer in category 3 requires posting and/or barricading to indicate AREA UNSAFE. Explain "yes" findings and extent of damage under "comments" then summarize all findings on page 1.

Condition	Yes	No	Comments
1. Structure Hazardous Overall			
Collapse/Partial Collapse	☐	☐	
Building or Story Leaning	☐	☐	
Other _____	☐	☐	
2. Hazardous Structural Elements			
Foundations	☐	☐	
Roof/Floors (Vertical Loads)	☐	☐	
Columns/Pilasters/Corbels	☐	☐	
Diaphragms/Horizontal Bracing	☐	☐	
Walls/Vertical Bracing	☐	☐	
Moment Frames	☐	☐	
Precast Connections	☐	☐	
Other _____	☐	☐	
3. Nonstructural Hazards			
Parapets/Ornamentation	☐	☐	
Cladding/Glazing	☐	☐	
Ceilings/Light Fixtures	☐	☐	
Interior Walls/Partitions	☐	☐	
Stairs/Exits	☐	☐	
Electrical	☐	☐	
Chemical Spills	☐	☐	
Radiological	☐	☐	
Other _____	☐	☐	
4. Geotechnical Hazards			
Slope Failure	☐	☐	
Ground Movement/Fissures	☐	☐	
Other _____	☐	☐	

ATTACHMENT A

~~HNF-SD-GN-ER-503, Rev. 0~~

APPENDIX A

INSPECTION GUIDELINES FOR STRUCTURAL SYSTEMS

The structural system of every building can be thought of as consisting of two systems:

1. Vertical-load-carrying system.
2. Lateral-load-carrying system.

For a structure to be considered as safe, both systems must be functional, and neither can be seriously degraded. This can be established either by inspecting the structural system directly, which may require removal of ceilings, walls, and other architectural finishes to permit viewing, or by careful inspection of different architectural finishes that will verify whether destructive building distortions have taken place. The latter approach is best used when serious structural damage is not suspected.

In order to perform a successful safety evaluation, it is crucial to be able to first identify the type of structural system, and then inspect those places in it where damage is likely to occur. The following pages present information describing the different types of building structural systems at the Hanford site and important structural aspects that must be assessed in evaluating the safety of a building. Building types are described in Table A1.

BUILDING TYPE	
W	Wood buildings of all types
S1	Steel moment resisting frames
S2	Braced steel frames
S3	Light metal buildings
S4	Steel frames with cast-in-place reinforced concrete shear walls
S5	Steel frames with unreinforced masonry infill walls
C1	Concrete moment resisting frames
C2	Concrete shear walls
C3	Concrete frames with unreinforced masonry infill walls
P1	Precast concrete tilt-up buildings
P2	Precast concrete frames
RM	Reinforced Masonry buildings
URM	Unreinforced masonry buildings

TABLE A1. BUILDING TYPE

WOOD FRAMED BUILDINGS

Wood framed structure design can vary and is generally based on the size of the building and clear span distance required for rooms inside the building. At the Hanford site, wood framed structures can be classified into two different types of wood framing that can be described as follows:

1. Conventional Light Framing
2. Structural Timber Construction

Each of these systems is described in more detail on the following pages along with guidelines for assessing structural distress.

Conventional Light Framing

Conventional light framing is characterized by stud framed bearing walls located at interior and exterior walls. Openings in these walls are made by replacing several studs with columns at each side of the opening and with a header beam across the top of the opening. In the case of a window opening, a short studded cripple wall is made for framing of the bottom of the opening. Exterior plywood sided walls normally act as shear walls to provide the lateral resistance of the structure. Roofs of these structures usually are made up of a plywood diaphragm attached to closely spaced trusses that are supported by exterior and interior bearing walls. These bearing walls may be attached directly to the foundation, but many times, a short studded cripple wall exists between the foundation and the main floor of the building.

Because wood is a tough, resilient material, with good energy-absorbing characteristics, the main structure has historically performed well. However, wood framed structures are generally flexible with resulting greater potential for damage to non-structural appendages. Connections are an important element in the survivability of wood structures.

Table A-1 lists suggested areas to examine for conventional light wood framed structures.

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Table A-1

INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
CONVENTIONAL LIGHT FRAMED WOOD BUILDINGS

Structural Condition -	Description	Posting
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Severe lean.....	UNSAFE
	Separation of Two Parts of a Building..	UNSAFE
	Movement off its Foundation.....	UNSAFE
2. Roof and Floor Framing	Roof or Floor Displaced from Walls.....	UNSAFE
	Failure or Incipient Failure of Significant Vertical Load-Carrying Element or Connection.....	UNSAFE
3. Walls	Residual Racking of 1 to 2 Inches or More per Story.....	UNSAFE
	Cripple Wall Failure.....	UNSAFE
4. Foundation Anchorage	Building is Off Foundation or Shows Other Signs of Movement.....	UNSAFE

~~WHC-SD-GN-ER-503, Rev. 0~~Structural Timber Construction

Wood structures that require wide and tall open interior areas are typically constructed differently than conventional light framed structures. These can be classified as structural timber construction. There is a great variety and diversity in this type of construction and it would out of the context of this procedure to try to cover all of them. However, many of them have basic similarities in design and can be briefly described as follows. The buildings typically have tongue-and-groove or 1x6 board siding or exterior plywood shear walls constructed with large timber columns spaced at approximately 20 feet with studded infill walls between the columns. The main vertical loads are transferred to the columns by large girders or trusses that may span 40 to 50 feet and may be supported on interior wood or steel columns.

The major problem with large wood framed structures is generally due to failure of a connection between the large members that define the basic structure. Also, as a result of its size, large openings may exist in the outer shear walls that leave the wall under-reinforced. Table A-2 lists inspection requirements for structural timber construction. Because of necessity, the guidance given below is very general.

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Table A-2

INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
STRUCTURAL TIMBER CONSTRUCTED BUILDINGS

Structural Condition -	Description	Posting
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Building or Individual Level Noticeably Leaning.....	UNSAFE
	Severely Racked Walls.....	UNSAFE
	Broken Foundation.....	UNSAFE
2. Roof, Heavy Framing	Failed Member or Connection in Roof Structural System.....	UNSAFE
	Heavy Framing Separation from Vertical Support.....	UNSAFE
	Other Failure or Incipient Failure of Significant Vertical Load-Carrying Element or Connection.....	UNSAFE
3. Roof/Floor-to-Wall Connection	Roof/Floor Framing has Pulled Away from Vertical Support.....	UNSAFE
4. Columns	Columns Noticeably Out of Plumb.....	UNSAFE
	Buckled or Failed Columns.....	UNSAFE
	Failed Anchorage Connection of Column to Foundation.....	UNSAFE
5. Wood Walls	Residual Racking of 1 to 2 Inches or More per Level.....	UNSAFE
	Failed Shear Wall or Failed Piers (In Plane) in Walls with Many Openings.....	UNSAFE
	Failure or Imminent Failure of Wall Providing Vertical Support.....	UNSAFE
6. Diaphragms	Bowed, Broken, or Seriously Damaged Diaphragms.....	UNSAFE
	Broken or Severely Damaged Chord or Collector Members.....	UNSAFE
	Movement or Failure at Shear Connection Between Diaphragm and Wall.....	UNSAFE
	Separated Sheets or Framing in Plywood Diaphragm.....	UNSAFE
7. Foundations	Building Off its Foundation or Shows other Signs of Movement.....	UNSAFE
	Fractured Foundations.....	UNSAFE

STEEL FRAMED STRUCTURES

Steel framed structures are structures whose main framing members are composed of steel shapes. These make up the portion of the structure that will carry most or all of the vertical load. Lateral forces are typically resisted by bracing, moment frame action, concrete or masonry shear walls, or a combination of these.

For purposes of facilitating the earthquake inspection guideline of damaged steel framed structures, steel framed structures are divided into the following classifications based primarily on the type of lateral-force-resisting system used.

1. Braced Frame
2. Moment-Resisting Frame
3. Prefabricated Metal Building
4. Frame with Unreinforced Masonry Infill Walls
5. Frame with cast-in-place Concrete or Reinforced Masonry Walls

Each of these systems is described in more detail on the following pages along with guidelines for assessing structural distress.

Braced Steel Frames

Bracing is used to transfer lateral forces imposed on a structure to the foundation. Vertical bracing may be X-, K-, or chevron-bracing and is usually present in the outer perimeter walls of the structure. Horizontal diaphragm bracing is also typically used at different levels or floors of a structure including the roof. Bracing is normally connected by bolts or welds to gusset plates that in turn are connected by bolts or welds to the beams, columns and base plates of the structure. The bracing may be any steel shape (i.e., angle, tee, I-beam). An important indicator of structural distress in braced structures is residual lean. Inspection guidelines and requirements for braced steel frames are combined with the same requirements for moment-resisting frames below in Table A-3.

Moment-Resisting Frames

With moment resisting frames, lateral forces are resisted by bending of the main structural members. Steel wide-flange columns and beams are typically connected by welded joints, although older structures may have bolted or riveted connections. Many times, this type of structural system is used in combination with braced frames where one scheme is used in one direction, and the other is used in the orthogonal direction. Under severe earthquake motion, moment-resisting frames may experience appreciable sidesway without collapse. The story drifts may be on the order of several inches, with accompanying racking of walls, and severely damaged structures may have residual story drift. Inspection guidelines and requirements for moment-resisting frames are combined with the same requirements for braced steel frames below in Table A-3.

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Table A-3

INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
BRACED STEEL AND MOMENT RESISTING FRAMES

Structural Condition -	Description	Posting
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Building or Individual Level Noticeably Leaning.....	UNSAFE
2. Roof and Floor Framing	Broken or Buckled Member in Truss.....	UNSAFE
	Failed Connection in Truss.....	UNSAFE
	Roof or Floor Framing Separation from Vertical Support.....	UNSAFE
	Other Failure or Incipient Failure of Significant Vertical Load-Carrying Element or Connection.....	UNSAFE
3. Columns	Columns Noticeably Out of Plumb.....	UNSAFE
	Buckled or Failed Columns.....	UNSAFE
	Failed Anchorage Connection of Column to Base or Foundation.....	UNSAFE
4. Vertical Bracing	Broken Brace or Connection.....	UNSAFE
	Buckled or Greatly Stretched Brace.....	UNSAFE
5. Moment Frames	Broken, Leaning, or Seriously Degraded Moment Frames.....	UNSAFE
	Weld or Other Connection Failure at Moment Joint.....	UNSAFE
	Flange Buckling near Joints.....	UNSAFE
6. Horizontal Bracing	Broken Brace or Connection.....	UNSAFE
	Buckled or Greatly Stretched Brace.....	UNSAFE
	Failed Chord or Connection.....	UNSAFE
7. Horizontal Diaphragms	Bowed, Broken or Seriously Damaged Diaphragm.....	UNSAFE
	Broken or Seriously Damaged Chord or Collector.....	UNSAFE
	Movement or Failure at Shear Connection Between Diaphragm and Beam.....	UNSAFE
	Horizontal Cracks 1/2 Inch Wide or Larger in Concrete Structural Slabs....	UNSAFE
8. Foundations	Building off its Foundation or Shows other Signs of Movement.....	UNSAFE
	Fractured Foundations.....	UNSAFE

~~WMC-SD-GN-ER-505, Rev. 0~~Prefabricated Metal Buildings

Prefabricated metal buildings are normally one-story, relatively lightweight structures. Typically, lateral forces are resisted by bents designed for moment-frame action in one direction and rod X-bracing with turnbuckles in the other. Framing is generally steel throughout, usually with metal decking and siding over light-gauge purlins and girts. Roofs normally have rod X-bracing with turnbuckles for adjustment. These buildings usually perform good from the standpoint of avoiding collapse, primarily as a result of being lightweight and are stiffened by the metal, nonstructural siding. Inspection guidelines and requirements are shown below in Table A-4.

Table A-4

INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
PREFABRICATED METAL BUILDINGS

Structural Condition -	Description	Posting
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Residual Story Drift of 2 Inches or More (in 10 Feet).....	UNSAFE
	Severe Twisting of Building.....	UNSAFE
2. Wall Bracing	Rod or Connection Failure.....	UNSAFE
	Broken Rods or Greatly Stretched Rods..	UNSAFE
3. Roof Bracing	Rod or Connection Failure.....	UNSAFE
	Sever Racking of Decking.....	UNSAFE
4. Steel Bents	Buckled Flange in Bent.....	UNSAFE
	Failed Bolted Moment Connection.....	UNSAFE
	Base Plate with Sheared Anchor Bolts...	UNSAFE
	Frame Out of Plumb 1 to 2 Inches or More.....	UNSAFE
5. Foundation	Fractured Foundation.....	UNSAFE

*Static force on
No sign of load transfer
Significant movement in floor joists - check down*

~~WHC-SD-GN-ER-503, Rev 0~~Frame with Masonry Infill Walls

The way a frame with masonry infill walls is designed to resist lateral seismic forces is not always readily apparent. Also, whether the masonry is reinforced or unreinforced may be hard to determine visually. The steel frame may be designed to resist 100 percent of all seismic loads imposed, with infill walls neglected structurally and used for architectural, security, or fireproofing reasons. Other frames may be designed initially to act as a shear wall structure and then as a moment-frame structure, provided the walls become sufficiently damaged. As a result of the masonry walls vulnerability to seismic induced damage, and for the inspector to assess the overall structural integrity of a building, unreinforced masonry infill walls should not be relied on to resist lateral seismic loads.

Because of the masonry infill, the structure tends to be very stiff. In major earthquakes, the infill walls may suffer substantial cracking and deterioration, thus reducing their stiffness. This in turn puts additional demand on the frame. Some of the walls may fail while others remain intact, which may result in torsion or soft story problems. In addition to structural concerns, the hazard from falling masonry is very significant. Inspection guidelines and requirements for inspection of masonry infill walls are shown in Table A-5. For additional guidelines addressing the steel portion of the frame, see Table A-3.

Table A-5

INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
MASONRY INFILL WALLS

<u>Structural Condition -</u>	<u>Description</u>	<u>Posting</u>
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Walls or Columns Noticeably Leaning...	UNSAFE
2. Infill Wall Units	Substantial Cracking (1/16 inch or greater) in Unreinforced Masonry.....	AREA UNSAFE
	Obvious Falling Hazard.....	AREA UNSAFE

Steel Frame with Cast-In Place Concrete Shear Walls

This type of structural system is similar to that of the steel moment-resisting-frame in that a matrix of steel columns and girders is distributed throughout the structure. The joints, however, are not designed to be moment-resisting and only the beam webs will be connected to columns or walls, either directly with welds or with welded, bolted, or riveted end plate or angle connections. Seismic lateral forces are transmitted through the steel/concrete wall connection and the concrete acts as a shear wall. The shear walls are normally located at an elevator core, service core, stairwell, interior or exterior wall. This building type also applies to steel framed with reinforced masonry shear walls. In the case of a severe seismic event resulting with severe structural damage to reinforced masonry walls, the hazard from falling masonry is significant.

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Normally, shear wall buildings are very stiff and resist collapse reasonably well. Building displacements during an earthquake will be smaller resulting in less potential for damage to non-structural appendages. See Table A-3 for guidelines in inspecting the steel portion of the frame and Table A-5 for inspecting a masonry shear wall. Refer to the sections on concrete for inspecting a concrete shear wall.

CONCRETE STRUCTURES

Concrete structures may be cast-in-place, precast, or a combination of the two. For both types of structures, the configuration of the concrete building can vary considerably. For the purposes of facilitating the earthquake inspection guideline of damaged concrete structures, concrete structures are divided into the following common types based on lateral-force-resisting system used.

1. Moment-Resisting-Frame
2. Infill Masonry
3. Shear Wall
4. Precast Frame
5. Tilt-up

Many concrete structures have mixed systems, frequently frame and shear wall combinations. Rather than create damage inspection guidelines for each system separately, all systems will be handled together. First, though, each lateral-force-system with their own inherent problems shall be discussed. Inspection guidelines and requirements for cast-in-place concrete structures follow in Table A-6. Table A-7 lists separate inspection guidelines and requirements for precast structural systems.

Concrete Moment Resisting Frames

Concrete moment resisting frame systems can be categorized as either ductile or nonductile. Some of the main differences are that ductile frames have a great many more columns ties and beam stirrups to confine the concrete and promote ductility. In addition, member proportions, reinforcing steel yield strength characteristics, and anchorage of reinforcement are important factors. The result is that ductile frames may sustain higher deflections and still maintain satisfactory load-carrying performance. However, these types of concrete structures are relatively new and typically used in higher seismic zones than at Hanford.

The more common type of concrete moment resisting frame at Hanford would be considered as nonductile. Some typical problems are:

1. Large tie spacings in columns can lead to a lack of concrete confinement and/or shear failure.
2. Placement of inadequate rebar splices at the same location can lead to column failure.
3. Insufficient shear strength in columns can lead to shear failure prior to the development of moment hinge capacity.
4. Insufficient shear tie anchorage can prevent the column from developing its full shear capacity.

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5. Lack of continuous beam reinforcement can result in hinge formation during load reversal.
6. Inadequate reinforcing of beam-column joints or location of beam bar splices at columns can lead to failures.
7. The relatively low stiffness of the frame can lead to substantial non-structural damage.

Concrete Frames with Masonry Infill

Concrete frames with masonry infill are usually older in design than concrete moment resisting frames and, as a result, the amount of and detailing of reinforcement is more in question. The hazards of these buildings are similar to and perhaps more severe than those of the newer concrete frames. As with steel frames with masonry infill, the problems encountered as to whether the masonry is reinforced or not exists. Where unreinforced masonry infill is present, a falling hazard exists. Failure and signs of distress of masonry infill in concrete frames is the same as for masonry infill in steel frames.

Concrete Shear Wall Structures

Cast-in-place concrete shear wall structures generally perform better than concrete frame buildings with much smaller story drifts. They are quite heavy relative to concrete frame structures but they are also very rigid due to the presence of the shear walls. Normally, the shear walls are poured to the level below the next floor, then the floor is poured, and then the next portion of the wall. Therefore, the quality of the construction joint between the walls and the floor becomes a major factor in seismic resistance and performance. Common areas of damage specific to this type of building, and should be closely inspected for, are:

1. Shear cracking and distress can occur around openings in concrete shear walls during large seismic events.
2. Shear failure can occur at wall construction joints at lower load levels than would be expected.
3. Bending failures can result from insufficient chord steel lap lengths.
4. Spalling of concrete and exposure of reinforcement can occur at nonconfined boundary elements.

Precast Concrete Frame Structures

Precast concrete buildings are constructed with precast concrete elements connected together by welded steel connections or poured-in-place concrete closure strips. Structural elements vary and beams and floor panels are often T's, double T's, or rectangular sections. Prestressing of the members, including pre-tensioning and post-tensioning, is often employed. Lateral forces are often resisted by diaphragms consisting of precast floor planks, with a cast-in-place floor topping, transferring loads to cast-in-place shear walls. Still, other buildings may have a precast frame structure to resist the lateral loads.

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The principal seismic concerns with precast structures are the joints and elements. Often the joints do not have the necessary strength and ductility to withstand a severe seismic event without damage or separation. Some of the problem areas specific to precast frames that require close inspection for, are:

1. Poorly designed connections between prefabricated elements can fail.
2. Accumulated stresses can result due to shrinkage and creep and due to stresses incurred in transportation.
3. Loss of vertical support can occur due to inadequate bearing area and/or insufficient connection between floor elements and columns.
4. Corrosion of metal connectors between prefabricated elements can occur.

Inspection guidelines and posting requirements for cast-in-place concrete are shown below in Table A-6.

Table A-6		INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR CAST-IN-PLACE CONCRETE BUILDINGS
Structural Condition -	Description	Posting
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Building or Individual Story Noticeably Leaning.....	UNSAFE
2. Slabs and Beams	Separation from Vertical Support.....	UNSAFE
	Other Failure or Incipient Failure of Significant Vertical Load-Carrying Element or Connection.....	UNSAFE
3. Flat Slab Construction	Slab with Punching Shear Cracking or Failure at Columns.....	UNSAFE
4. Columns	Buckled or Fractured Column.....	UNSAFE
	Massive Spalling and Exposure of Vertical Reinforcement.....	UNSAFE
	Large Diagonal Crack Extending through Column.....	UNSAFE
5. Moment Frames	Seriously Degraded Moment Frames.....	UNSAFE
	Severe Panel Zone Cracking.....	UNSAFE
	Noticeable Residual Drift in Any Story.	UNSAFE
6. Horizontal Diaphragms	Broken Floor-Wall Connection.....	UNSAFE
	Broken or Seriously Damaged Diaphragm..	UNSAFE

Table A-6
(Continued)INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
CAST-IN-PLACE CONCRETE BUILDINGS

Structural Condition -	Description	Posting
7. Solid Shear Walls <i>1/2" still has steel all around being still there</i>	Diagonal (Shear) Cracks of 1/8 Inch Wide or More Extending Between Floors..	UNSAFE
	Slippage 1/8 Inch Wide or More at Horizontal Construction Joint.....	UNSAFE
	Spalling of Concrete and Exposure of Vertical Reinforcement at Boundary Elements.....	UNSAFE
	Horizontal Cracks of 1/8 Inch Wide or More Extending through Boundary Elements.....	UNSAFE
8. Shear Walls with Openings	Several Failed Piers at Any One Story..	UNSAFE
	Failed Spandrel Beams.....	UNSAFE
9. Infill Masonry Walls	Infill Wall Failure.....	UNSAFE
	Large Infill Wall Crack Extending through Concrete Frame.....	UNSAFE
	Infill Wall Failure along Horizontal Bedding Plane.....	AREA UNSAFE
10. Foundations	Bowing of Underground Walls.....	UNSAFE
	Fractured Foundations.....	UNSAFE

Inspection guidelines and posting requirements specific to precast concrete structures are given below in Table A-7

Table A-7

INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
PRECAST CONCRETE BUILDINGS

Structural Condition -	Description	Posting
1. Beam-Column and Other Member Connections	Failed Joints of Primary Members.....	UNSAFE
	Corbel or Other Cracking that Threatens Vertical Support.....	UNSAFE
	Massive Spalled Concrete in Closure Strip.....	UNSAFE

Concrete Tilt-Up Structures

Concrete tilt-up structures consist of sitecast or precast concrete panels that are lifted or tilted into place. The wall panels are welded together or held in place by cast-in-place concrete columns or steel columns. Panels may be load bearing with floor or roof systems attached to them and normally are located around the perimeter of the building and act as shear walls. Roof construction may either be wood framing with plywood diaphragms or steel framing with metal deck. Common areas of damage specific to this type of building that require close inspection for, are:

1. Roof or floor collapse can occur as a result of weak anchorage ties to tilt-up panels.
2. Building distortion with large gaps between concrete panels can occur as a result of failure connection between panels.

Table A-8

INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
CONCRETE TILT-UP BUILDINGS

<u>Structural Condition -</u>	<u>Description</u>	<u>Posting</u>
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Building Noticeably Leaning.....	UNSAFE
2. Roof and Floor Framing	Failed Member or Connection in Roof Truss.....	UNSAFE
	Roof or Floor Framing Separation from Vertical Support.....	UNSAFE
	Other Failure or Incipient Failure of Significant Vertical Load-Carrying Element or Connection.....	UNSAFE
3. Horizontal Diaphragms	Broken or Seriously Damaged Diaphragm..	UNSAFE
	Severely Damaged Chord or Collector....	UNSAFE
	Movement or Failure at Shear Connection Between Diaphragm and Wall Panels.....	UNSAFE
4. Columns	Columns Noticeably Out of Plumb.....	UNSAFE
	Buckled or Failed Columns.....	UNSAFE
5. Wall Panels	Outward Leaning Wall Panel.....	UNSAFE
	Severely Damaged Closure Strips, or a Number of Broken Tie Plates.....	UNSAFE
	Wall Instability Resulting from Cracking of Panels.....	UNSAFE
	Wall, Corbel or Pilaster Cracking That Jeopardizes Vertical Support of Beam or Truss.....	UNSAFE
6. Foundations	Fractured Foundations.....	UNSAFE

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MASONRY BUILDINGS

Masonry structures, as opposed to masonry infill wall structures, rely on the masonry walls to carry vertical loads as well as resist lateral forces. Masonry structures can be categorized as follows:

1. Reinforced
2. Unreinforced

Seismic performances of each type varies dramatically. Following, is a discussion of each type of structure depicting inherent structural deficiencies. A list of inspection guidelines and posting requirements, combined for both systems, is given in Table A-9.

Reinforced Masonry Buildings

Reinforced masonry buildings typically have perimeter shear walls. Vertical loads can be carried on pilasters or by direct bearing on walls. The walls consist of grouted and reinforced hollow or solid masonry units. Concrete masonry units (CMU) are most common. Typically, concrete beams or columns are not present except for concrete framing around openings or beneath concentrated loads. Floor and roof assemblies usually consist of wood framing with plywood diaphragms, steel framing with metal deck, with or without concrete fill, cast-in-place concrete, or precast concrete planks with a concrete topping. Interior supports, if any, are often wood or steel columns, wood stud frames, or solid masonry units. It is often hard to distinguish whether the masonry is reinforced or not and a review of drawings for reinforcing details may be necessary. However, large reinforced masonry buildings are characterized by the presence of control joints to control shrinkage and temperature-related cracking. Reinforcement generally does not extend through the control joint.

Seismic performance of reinforced masonry buildings can be generally good. Problems are encountered when there is inadequate reinforcement or insufficient positive connection of the floor/roof diaphragm(s). Other problems that are common include poor workmanship during construction or deterioration of the mortar in the joints due to years of exposure to the weather.

Unreinforced Masonry Buildings

Unreinforced masonry buildings are typically not distinguishable from reinforced masonry buildings and a review of drawings for reinforcement is necessary. Structural systems for unreinforced masonry buildings are similar to reinforced masonry except that unreinforced bearing walls will tend to be thicker at the base and taper off in thickness throughout the height of the building.

Unreinforced masonry structures are probably the most hazardous form of construction and those structures that have not collapsed after a strong earthquake usually will require demolition. Unreinforced masonry buildings typically have not been used at Hanford.

Table A-9

INSPECTION GUIDELINES AND POSTING REQUIREMENTS FOR
REINFORCED AND UNREINFORCED MASONRY BUILDINGS

Structural Condition	Description	Posting
1. Overall Damage	Collapse or Partial Collapse.....	UNSAFE
	Building or Individual Story Noticeably Leaning.....	UNSAFE
2. Roof and Floor Framing	Roof or Floor Framing Separation from Wall.....	UNSAFE
	Pilaster Cracking.....	UNSAFE
	Other Failure or Incipient Failure of Significant Vertical Load-Carrying Element or Connection.....	UNSAFE
3. Reinforced Masonry Walls	Walls with 1/8 Inch or Larger Diagonal (Shear) Cracks.....	UNSAFE
	Several Failed Piers at Any One Story Level.....	UNSAFE
	Walls with Out of Plane Leaning.....	UNSAFE
	Wall or Pilaster Cracking or Spalling that Jeopardizes Vertical Support.....	UNSAFE
	Walls with Falling Hazards.....	AREA UNSAFE
4. Unreinforced Masonry Walls	Walls Out of Plumb.....	UNSAFE
	Diagonal or Other Large Cracking.....	UNSAFE
	Wythe Separation and Masonry Spalling..	UNSAFE
	Cracking that Jeopardizes Vertical Support.....	UNSAFE
5. Horizontal Diaphragms	Bowed, Broken or Seriously Damaged Diaphragm.....	UNSAFE
	Broken Chord or Collector.....	UNSAFE
	Movement or Failure at Shear Connection between Diaphragm and Masonry Wall.....	UNSAFE
6. Foundations	Fractured Foundations.....	UNSAFE
	Large Differential Settlements or Other Signs of Ground Movement.....	UNSAFE

ATTACHMENT B

Concrete and Reinforced Masonry

- ▶ Repair of cracks $> 1/64"$ recommended by ACI for environmental reasons
- ▶ Cracks up to 0.25" repair by epoxy.
If larger, use cement grout
- ▶ Concrete crushing is the primary indication of a flexural mechanism.
- ▶ Cracks greater than 0.25 in width in usually indicate a shear deformation mechanism.
- ▶ For major crushing and large cracks, the steel reinforcement should be inspected to evaluate whether it has been fractured or buckled.

5.

Evaluation of Reinforced Concrete Members Damaged by Earthquakes

Robert D. Hanson, M.EERI

ABSTRACT

A number of building authorities have included or are proposing to include loss in lateral capacity of the structural system caused by earthquake damage as a basis for requiring specific degrees of seismic repair and upgrades of the damaged members or of the entire structural system. Attempts have been made to apply this criteria through the size of cracks in reinforced concrete walls. This paper reviews experimental results which demonstrate that size of wall crack is not directly related to a reduction in wall capacity. The effectiveness of various wall crack repair techniques on restoring wall characteristics is discussed.

INTRODUCTION

A number of building authorities have included or are proposing to use a reduction in lateral capacity due to earthquake caused structural damage as a basis for requiring specific degrees of seismic upgrade of the damaged members or the total building. Although these requirements assume or are interpreted to assume that lateral capacity means lateral ultimate strength, it is important to recognize that the performance of buildings during earthquake ground motions depends upon a number of factors including ultimate strength, stiffness, deformability, stability, and energy dissipation (or absorption) capacities. Even when the proposed requirements limit their consideration to ultimate strength capacity, the proposers also assume that a rational basis is available for the determination of a loss in ultimate lateral strength capacity. From reviewing several attempts to make this assessment of capacity loss, it is clear that these members of the structural engineering profession have not been successful in achieving this goal. These attempts have proposed a relationship of the loss in ultimate strength capacity to the widths of the cracks and their horizontal projection in damaged reinforced concrete members.

The purpose of this paper is to explore the relationship of crack occurrence, and their widths, to loss in ultimate strength capacity of reinforced concrete walls, and to explore what might provide a common basis on which to estimate the loss in structural ultimate strength

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capacity caused by earthquake damage of reinforced concrete walls. It will be shown that cracks effect the stiffness of these elements, but that crack widths do not provide a rational basis for the determination of the lateral strength capacity reduction until the cracks become significantly large. The repair of cracked reinforced concrete elements to provide environmental protection (corrosion prevention), stiffness enhancement and for structural strength development are discussed.

This paper does not address schemes for upgrading structural systems damaged by earthquakes to higher design lateral force levels, nor the efficacy or benefit/cost of particular lateral force resisting system structural upgrade schemes.

TOLERABLE CRACKS

American Concrete Institute Committee 224 has reported a number of times on causes, evaluation and repair of cracks in concrete (ACI 1980, 1984). Among causes of cracks they include (a) plastic shrinkage, (b) settlement, (c) drying shrinkage, (d) thermal stresses, (e) chemical reactions, (f) weathering, (g) corrosion of reinforcement, (h) poor construction practices, (i) construction overloads, (j) errors in design and detailing, and (k) externally applied loads (ACI 1984). From these many factors it is sometimes difficult to identify the appropriate combination of causes for some cracks. In fact, externally applied excitations, such as earthquakes, may in some instances only activate or cause movement of cracks caused by other factors.

In the case of repeated large external loads, such as different earthquakes over the life of a building, the cracks may originate during the first loading event and only reactivate during succeeding events many years later. It is often difficult to determine whether the latest loading event caused a new crack or reactivated a pre-existing crack unless the earlier crack had been repaired or painted prior to the current external loading and the plaster or paint had penetrated the crack.

Table 1 repeats the tolerable crack widths in reinforced concrete to be considered in the design process (ACI 1980). These crack widths are given for structures located in many environmental settings. This is to illustrate that it is clearly understood that concrete cracks of these sizes are normal for well designed buildings. It should be noted that the environmentally tolerable crack for interior building areas used for human occupancy is 0.016 in (0.41 mm).

IMPORTANCE OF CRACKS IN STRUCTURAL BEHAVIOR

Almost all of the experimental and analytical research efforts for earthquake resistant design have been made to develop equations for the design of reinforced concrete members which will achieve the desired performance during selected earthquake excitation. Post-earthquake investigations have primarily focused on identifying successes and shortcomings

TABLE 1

Tolerable crack widths, reinforced concrete [from ACI (1980) Table 4.1]

Exposure Condition	Tolerable crack width, in.	Tolerable crack width, mm
Dry air or protective membrane	0.016 $\approx \frac{1}{64}$	0.41
Humidity, moist air, soil	0.012	0.30
Deicing chemicals	0.007	0.18
Seawater and seawater spray: wetting and drying	0.006	0.15
Water retaining structures*	0.004	0.10

*Excluding nonpressure pipes

of the design approaches used for the specific structural system, and not to establish a basis for judging loss in stiffness or strength caused by the earthquake. That is, the observed earthquake damage is seldom related to earthquake strength and stiffness. Although the following discussion is intended to lead to such an assessment, it necessarily mixes design and performance aspects.

Forces in structural members are classified into three types: axial force, bending moment, and shear force. In reinforced concrete members, the concrete and steel reinforcement share in the resistance to these forces primarily with the concrete acting in compression and with the steel providing the required tensile capacity. The usual engineering practice, even for elastic (allowable stress) design, is to assume that the strength of concrete in tension is zero. For example, it is commonly recognized by the building design profession that concrete cracks are a natural part of the behavior of reinforced concrete members. Even in elastic allowable moment design, cracked sections are assumed in the determination of the beam flexural capacity. As stated above the desire is to keep the size of the cracks small enough to avoid material deterioration and, for visible members, to avoid unsightly cracks. The initial stiffness of these members is usually assumed to be a stiffness between the gross uncracked section stiffness and the fully cracked section stiffness.

In the case of capacity evaluations of cracked concrete members with a minimum amount of steel reinforcement anchored to develop the steel yield strength, it is important to note that the full strength of the member cannot be mobilized until after the steel yields, or until the concrete crushes. In order for forces to be transferred to the steel, tension cracks must occur in the concrete. For example, for flexural members, the limit state design procedure uses (1) full yield strength of the steel tensile reinforcement, (2) concrete in compression with a maximum strain of 0.003 in./in., and (3) compressive steel stress up to its yield value, depending upon its location.

As another example, in recently constructed special moment resisting concrete frame buildings, the flexural members are required to develop a positive moment capacity at the column face to be at least one-half of the negative moment capacity. This is to assure full cyclic capacity of the section. In extremely large cyclic yielding where the top and bottom bars have significant residual elongation or where significant concrete crushing has occurred, the flexural tension cracks can develop permanent residual concrete crack separation. Only when this concrete crushing and/or steel elongation occurs is the shear capacity at this cross section reduced to the shear force which can be transmitted by the steel alone. At this level of degradation, shear is carried by (1) the longitudinal reinforcing steel through dowel action, and (2) the steel ties which cross the crack.

The behavior of concrete walls in shear is somewhat more complicated because their performance is influenced by both (1) the amount of axial force and (2) the bending moment at the section. If pure shear without axial forces or bending moments is considered, the shear force will be carried by the horizontal and vertical reinforcement in the member, and the concrete will act as a compression strut. Park and Pauley (1975) state that the triangular free body diagram 1 in Figure 1 shows that the horizontal stirrups are needed to resist the shearing stress applied along the top edge. In the absence of overall wall axial compression, free body diagram 2 shows that the vertical reinforcement is necessary to generate the necessary diagonal compression in the concrete.

Figure 1. The Shear Resistance of Low-Rise Shear Walls
[from Park (1975), Figure 12.6, page 619]

In the case of shear walls, as the height of the wall increases relative to the length of the wall the behavior changes from nearly pure shear to nearly pure flexure in the absence of

axial compression loads. Axial compression loads, which exist when the shear walls serve as bearing walls, increase the capacity of these walls and can change the ultimate failure mechanism of the wall from flexural mechanism by increasing the flexural strength to values larger than the shear mechanism strength.

Current seismic design requirements for reinforced concrete flexural members are established to achieve flexural yielding rather than large shear deformations. This is accomplished by designing the member's shear capacity to resist probable flexural end moment overstrengths without a capacity reduction factor. For beams and columns this is accomplished by providing sufficient amounts of closely spaced ties around the longitudinal reinforcement. For walls the web reinforcement is selected based on the results of an elastic analysis. The wall capacity is accomplished by providing horizontal, vertical and, in some cases, diagonal steel reinforcement.

A large number of research studies have been performed to develop and refine the equations for designing members to achieve this performance. For walls these designs typically result in wall boundary members to resist both tensile and compressive flexural forces and horizontal and vertical reinforcement across the entire width of the wall to carry the corresponding shear forces. Wood (1991) has reviewed 27 laboratory tests of isolated reinforced concrete walls with aspect ratios (the height of the wall divided by the length of the wall) of 1.3 to 2.8 to determine the influence which (1) the amount of reinforcement, (2) the axial stress, and (3) the loading history have on their displacement capacity. Some of the tests failed in shear although they were predicted to fail in flexure. She reported that, based on these tests, the ACI 318-89 Code (ACI 1989) equations for *"the relative values of the nominal flexural and shear strengths were not good indicators of the mode of failure observed in the test specimens. Walls with calculated flexural strengths considerably less than the calculated shear strength were observed to fail in shear"* (Wood 1991).

Alternative calculations using formulas normally used for columns were also tried to predict the failure mode for these walls, with an equal lack of success, these calculations predicted all flexural failures. Nevertheless, Wood stated that *"Although the calculated nominal flexural and shear strengths are not good indicators of the likely mode of failure of walls subjected to cyclic loading ..., the nominal strength, defined as the minimum of the nominal flexural and shear strengths, appears to be a reasonable estimate of the lateral strength"* (Wood 1991). Even with careful selection of flexural and shear reinforcement it is not possible to assure that the wall failure will be flexural or shear type. Fortunately, the test results indicate that *"The mean drift sustained by the test specimens subjected to cyclic loading did not appear to be sensitive to the observed mode of failure ... Values of maximum drift ranged from 1.5 to 3.9 % for walls that failed in flexure and from 1.1 to 3.5 % for walls that failed in shear"* (Wood 1991). It should be noted that "drift" in this quotation refers to the interstory drift ratio.

The significance of Wood's observations is that while our ability to predict failure mechanisms of isolated shear walls is not good, our ability to judge their probable ultimate

capacities is reasonable, and that the hysteretic load-drift characteristics are not highly dependent upon the failure mechanism at large deformations.

Returning to the main purpose of this paper, which is to explore a potential basis for determining the loss in structural capacity of members damaged by earthquakes, a review of selected experimental tests will provide some insight to possible answers.

EXPERIMENTAL STUDIES

SLENDER WALLS

PCA Tests. In the 1970's the Portland Cement Association performed a series of tests on isolated shear walls with and without enlarged boundary members (Oesterle 1976). The test specimens were 180 in. high, 75 in. long, and had a web thickness of 4 in. (aspect ratio of 2.4). For this paper Specimens R1, B1 and B2 have been selected for discussion because they have unconfined boundary reinforcement, low reinforcement ratios, and very small axial forces. The amount of reinforcement in these walls ranged from 0.25 to 0.29 percent in the vertical direction and from 0.31 to 0.63 percent in the horizontal direction. The usual current minimum specified wall reinforcement is 0.25 percent each way, ACI 318-89 (ACI 1989), as compared with the 1950's usual minimum specified wall reinforcement of 0.20 percent each way. Test specimen properties and analysis of these and other test results are given by Wood (1991).

The load-deflection curves for the first 13 cycles of specimen R1 are given in Figure 2 and for the entire test in Figure 3. The performance of this test specimen was primarily flexural. The cracking load was reported to be 12.0 kips. The first yield load was 20.1 kips at a displacement of 0.43 in. with a corresponding crack width of 0.018 in. The maximum load was 26.6 kips at a displacement of 3.0 in. with a corresponding crack width of 0.200 in.

The load-deflection curves for the first 13 cycles of specimen B1 are given in Figure 4 and for the entire test in Figure 5. The performance was primarily flexural. The cracking load was 26.0 kips. The first yield load was 45.1 kips at a displacement of 0.56 in. and crack width of 0.014 in. The maximum load was 61.0 kips at a displacement of 4.0 in., and the crack width was not reported.

The load-deflection curves for the first 13 cycles of specimen B2 are given in Figure 6 and for the entire test in Figure 7. The performance at failure was primarily shear. The cracking load was 30.0 kips. The first yield load was 120 kips at a displacement of 0.84 in. with a corresponding crack width of 0.017 in. The maximum load was 153 kips at a displacement of 4.0 in., and the crack width was not reported.

It is clear from these force-deflection curves that the capacity of the walls are always larger than at steel yield load which in turn is always larger than the concrete cracking load. It is also clear that wall stiffness decreases after cracking and again after yielding. Because

Figure 2. First Thirteen Continuous Load-Deflection Cycles for Specimen R1
[from Oesterle (1976), figure B-8, page B-18]

Figure 3. Continuous Load-Deflection Cycles for Specimen R1
[from Oesterle (1976), figure B-9, page B-19]

Figure 4. First Thirteen Continuous Load-Deflection Cycles for Specimen B1
[from Oosterle (1976), figure B-51, page B-69]

Figure 5. Continuous Load-Deflection Cycles for Specimen B1
[from Oosterle (1976), figure B-52, page B-70]

Figure 6. First Thirteen Continuous Load-Deflection Cycles for Specimen B2
[from Oosterle (1976), figure B-112, page B-140]

Figure 7. Continuous Load-Deflection Cycles for Specimen B2
[from Oosterle (1976), figure B-113, page B-141]

the crack sizes and distribution are a function of the reinforcement ratio and reinforcement distribution and because crack sizes do not affect the capacities of these members, very few crack measurements have been taken and reported with the experimental results.

Pilakoutas and Elnashai Tests. Recent experimental results of isolated reinforced concrete walls with boundary members (Pilakoutas 1995) illustrate the performance of walls with combined flexure-shear behavior. These 1/2.5 scale walls with an aspect ratio of two had a height of 1200 mm (47.32 in.), a length of 600 mm (23.62 in.), and a web thickness of 60 mm (2.36 in.). The notation used in their figures is as follows: the maximum displacement level, MDL-mm, gives the cyclic displacement single amplitude of that test cycle in millimeters and the SW# gives the specimen number.

In specimen SW4 (vertical web reinforcement ratio of 0.31 percent and horizontal web reinforcement ratio of 0.39 percent) first yield occurred just before MDL-6, that is, just before the cyclic test single amplitude displacement reached 6 mm (0.24 in.). Thus, in Figure 8 the cracks in MDL-4 occurred before first yielding. They report that by MDL-16 the web cracks in the lower portion of the wall had opened up considerably more than the others. The load-top displacement behavior is given in Figure 9.

Specimen SW5 had more flexural capacity and smaller shear capacity than SW4. SW5 had vertical web reinforcement ratio of 0.47 percent and horizontal web reinforcement ratio of 0.31 percent. Yielding of the horizontal web reinforcement occurred just prior to attainment of yield in the main flexural reinforcement before achieving MDL-10. Thus, in Figure 10 the cracks at MDL-2 occurred long before first yielding. They report that at MDL-10 an abrupt failure occurred when trying to impose this displacement level in the reverse direction and that two of the main web cracks opened up significantly. Figure 11 shows the degradation in capacity for additional cycles after MDL-12. Nevertheless, the wall still retained about 1/3 of its maximum capacity at displacements of 24 mm (0.94 in.).

Specimen SW6 had a flexural capacity similar to SW4 and a shear capacity similar to SW5. SW6 had vertical web reinforcement ratio of 0.31 percent and horizontal web reinforcement ratio of 0.31 percent. Yielding of the main flexural reinforcement occurred just prior to MDL-6 and the extent of web cracking increased considerably after this yielding. Following MDL-8 cracks were considerably wider than for the corresponding MDL for SW4. By MDL-18 concrete deterioration started to affect the wall strength and concrete crushing initiated just before MDL-20. The crack pattern and load-displacement curves are given in Figures 12 and 13.

Their conclusions included the observations that *"The strength and deformational characteristics of specimens were not affected significantly by shear reinforcement in excess of the amount required to resist the maximum applied load."* and *"Shear force is partly transmitted by the concrete in compression and partly by the link reinforcement that enables shear stresses to be resisted through the concrete in the tensile zone."*

Figure 8. Crack Pattern of Wall SW4 at MDL-4, MDL-8 and Failure
[from Pilakoutas (1995), figure 5, page 274]

Figure 9. Loads Versus Wall SW4 Top Displacement
[from Pilakoutas (1995), figure 6, page 274]

Figure 10. Crack Pattern of Wall SW5 at MDL-2, MDL-10 (Failure) and MDL-24 (Post-Failure) [from Pilakoutas (1995), figure 7, page 275]

Figure 11. Load Versus Wall SW5 Top Displacement [from Pilakoutas (1995), figure 8, page 275]

Figure 12. Crack Pattern of Wall SW6 at MDL-2, MDL-8 and Failure
[from Pilakoutas (1995), figure 9, page 275]

Figure 13. Load Versus Wall SW6 Top Displacement
[from Pilakoutas (1995), figure 10, page 276]

LOW-RISE SHEAR WALLS

Park and Pauley Tests. Park and Pauley (1975) researched the post elastic behavior of reinforced concrete shear walls by undertaking a series of tests on small walls 20 in. high, 20 in. long, and 6 in. thick (aspect ratio of one). Wall A was underdesigned for shear, with reinforcing sufficient to carry only 50 percent of the load assuming that flexure governed the strength; Wall B was the same as Wall A except that the horizontal shear reinforcement was provided in excess of the flexural capacity; and Wall C had more vertical reinforcement together with sufficient horizontal reinforcement to assure flexural failure (actual failure was a sliding shear failure). The failure modes and performance are illustrated in Figures 14 and 15. In

Figure 14 the wall cracking patterns are shown for the maximum displacements in various cycles. Figure 15 provides the cyclic load-displacement curves where cycles 1 to 4 show the initial elastic responses and cycles 7 to 8 show the elastic response following a significant cyclic excursion beyond yielding. Again all walls show capacities greater than their yielding capacity. The wall cracking patterns in Figure 14 can be correlated with the corresponding hysteretic load displacement history in Figure 15. It is observed that the lightly reinforced walls can develop large inelastic deformations without significant loss in load capacity and that extremely large cracks occur prior to reduction in load capacity.

Wood (1990). The results of 143 low-rise reinforced concrete walls tested in Canada, Japan, New Zealand, and the United States subjected to lateral loads are reviewed in this reference. The purpose of the study was to evaluate current ACI (1983) design provisions for nominal shear strength. The cross sections were symmetric of barbell, flanged and rectangular shapes. The horizontal web reinforcement ratios ranged from 0.07 to 1.9 percent and the vertical web reinforcement ratio ranged from 0.07 to 2.9 percent. Horizontal and vertical web reinforcement ratios differed by less than 10 percent in 90 percent of the walls. The longitudinal reinforcement ratios in the boundary elements varied from 0 to 8.9 percent.

The wall dimensions were given as a shear span ratio (Moment/shear load times the wall length) rather than as an aspect ratio. If the walls were simple cantilevers, the shear span ratio is the same as the aspect ratio. Shear span ratio was between 0.5 and 1.0 for more than 75 percent of the test specimens. Wood (1990) provides a number of different comparisons by utilizing test results for different test specimen parameter combinations. One comparison of most interest to this paper is the variation of the maximum shear strength divided by the square root of the concrete compressive strength, $\sqrt{f'_c}$, versus the amount of web reinforcement, Figure 16. The web reinforcement ratio is normalized by multiplying the reinforcement ratio by the measured steel yield stress. For example, a reinforcement ratio of 0.25 percent with a steel yield stress of 60,000 psi would have a normalized web reinforcement of 150 psi. Wood's conclusion from these evaluations is that the shear capacity of low-rise shear walls is relatively independent of the percentage of web reinforcement. Wood (1990) concluded that the ACI 318 shear equation *underestimates the nominal shear strength of lightly reinforced walls and may overestimate the nominal shear*

Figure 14. Failure Modes of Three Squat Shear Wall Models

(a) Wall A (b) Wall B (c) Wall C

[from Park (1975), figure 12.7, page 621]

Figure 15. Load-Rotation Relationships for Squat Shear Walls Subjected to Cyclic Loading. (a) Wall A (b) Wall B (c) Wall C
[from Park (1975), figure 12.8, pages 622-625]

strength of wall with reinforcement ratios greater than 0.38 percent. She recommends that the lower bound of the nominal shear capacity should equal $(6\sqrt{f_c})$ vs $10\sqrt{f_c}$ per ACI. As the amount of vertical reinforcement increased she recommends using a shear friction model in which the nominal shear strength increases linearly with the amount vertical reinforcement with an upper bound of $10\sqrt{f_c}$.

Figure 16. Variation of the ratio of measured shear strength to $\sqrt{f_c}$ with the amount of web reinforcement [from (Wood 1990), figure 6, page 103]

REPAIR OF CRACKS

Many studies have reported on various techniques to repair cracks in reinforced concrete. Some techniques are meant to seal the cracks to avoid reinforcement corrosion, others are to maintain/restore strength, stiffness and/or energy dissipation capacity. Hanson (1977) has provided a summary of various repair techniques which depend upon the size of the cracks or degree of damage to be repaired. It was recommended that cracks in all types of structural elements - beams, columns, walls and floor units - smaller than 0.25 in. be repaired using epoxy injection. It is stated that smaller cracks require higher injection pressures and/or more closely spaced ports, but the minimum size crack to be repaired is not given. For cracks larger than 0.25 in. or regions where the concrete has crushed, repairs using expansive cement mortar, Type III high-early strength mortar, or other materials should be

used to replace the highly fractured concrete. In severely damaged reinforced concrete members, it is possible that the reinforcement has buckled, experienced elongation with excessive yielding, or in the extreme case, may have fractured. This reinforcement can be repaired by replacement with new steel. If practical, the repair should be made without removing the existing steel.

Epoxy repair of concrete cracks has been shown by many tests to be fully effective in sealing the cracks. New cracks formed adjacent to the initial cracks during retesting. The biggest concern has been the restoration of bond between the reinforcing steel and the concrete in beam-to-column connections which had experienced bond failure during initial tests. French, Thorp and Tsai (French 1990) performed two test series to determine the effectiveness of epoxy techniques to repair moderate earthquake damage to reinforced concrete beam-column subassemblages. They compared pressure injection and vacuum impregnation repaired specimens with the original specimens. Cracks smaller than 0.005 in. (0.1 mm) were not repaired and the primary crack (0.25 in. or 6 mm) was not repaired well using the vacuum impregnation technique. Some minor concrete spalling was not repaired. They concluded "Both repair techniques worked well. The repaired structures achieved over 85 percent of the initial stiffness of the originally undamaged structures...the strength and energy-dissipation capacities were restored in the repaired specimens...The bond between the reinforcement and the concrete also appeared to be restored" (French 1990). Rodriguez and Park (1991) have provided a recent paper which provides a review of the literature on the repair and strengthening of reinforced concrete buildings in seismic areas with an emphasis on strengthening reinforced columns.

Since the strength of shear walls without concrete crushing and with cracks less than or equal to 0.25 in. do not have a loss in strength, the purposes of the epoxy repairs are intended only to (1) provide corrosion protection, (2) restore some of the initial stiffness of the wall, and (3) restore some of its energy dissipation capacity. Lefas and Kotsovos (1990) studied the effects of repairing reinforced concrete walls using 1/2.4 scale specimens with an aspect ratio of two. The wall height was 1300 mm (51.18 in.), wall length was 650 mm (25.59 in.), and wall web thickness of 65 mm (2.56 in.). The web horizontal reinforcement ratio was 0.35 percent and the web vertical reinforcement ratio was 1.5 percent. Two repair methods were used. Method A involved only the replacement of the damaged concrete in the compressive zone, and Method B also included epoxy resin injection into the major tension and the inclined cracks in the wall web larger than 0.2 mm (0.008 in.). They found that "the experimentally obtained strength (of both the original and repaired specimens) exceeded by a significant margin the values of horizontal load corresponding to either flexural capacity or shear strength (assuming that the concrete contribution to shear in the plastic hinge region subjected to load reversals is considered equal to zero)." (Lefas 1990). From these tests the authors observed that wall cracks appeared when the lateral load was about 15 percent of the wall capacity, as the applied load neared 60 percent of its capacity significant inclined cracks penetrated deeply into the web near the compression zone, and that these cracks formed an orthogonal crisscrossing pattern. For the specimens repaired by Method A, the cracking pattern was identical to that of the original specimens. For specimens repaired by Method B, they developed new cracks between the old repaired cracks. For loads beyond 80 percent of

the failure load (ultimate capacity), the crack pattern underwent insignificant changes. Lefas (1990) concluded that *"The load carrying capacity of the repaired walls depends mainly on the loading history of the reinforcing steel and is essentially independent of concrete strength...the repaired walls exhibited lower stiffness and less ductility than the original ones. Although the epoxy resins appears to have an insignificant effect on the ultimate strength of the walls, it is found to improve both stiffness and energy-dissipation characteristics of the walls at the serviceability limit state."*

64 Some building jurisdictions have criteria for repair techniques in which they accept without additional demonstration the repair effectiveness. Some of these guidelines specifically address minimum crack sizes for which epoxy injection can be used. Cracks as small as 0.002 in. (0.05 mm) can be bonded by the injection of epoxy (ACI 1984). However, from Table 1 it is clear that cracks on the interior of normal occupancy buildings smaller than 0.016 in. (0.41 mm) need not be repaired. A summary of these repair guidelines is beyond the purpose of this paper.

SUMMARY AND CONCLUSIONS

It is known that concrete cracks, that reinforced concrete members mobilize their strength by cracking, and that only cracks larger than those which can be tolerated for visual purposes or environmental protection of the members needs to be repaired.

From the studies referenced herein it is clear that (1) reinforced concrete structural members crack before the steel reinforcement yields, (2) these cracks increase in size without reduction in member load capacity, and (3) these members can sustain displacements far beyond the displacements at maximum capacity without excessive loss in load carrying capacity. The possible exception to this ability to maintain load carrying capacity far beyond the maximum capacity displacement is when the capacity is limited by early shear failure mechanisms. Even in this exceptional case the worst test data showed that about 1/3 of the maximum capacity was maintained at large displacements.

It is also clear that the stiffness of these members decrease as the displacements increase. The importance of any reduction in member stiffness depends upon the overall characteristics of the structural system. For shear wall buildings, this reduction in wall stiffness could result in a reduction in building stiffness with a corresponding reduction in earthquake forces and changes in story drifts. Each building must be evaluated for the consequence of reduction in member stiffness.

From this review of experimental test data, it must be concluded that measuring widths and lengths of reinforced concrete cracks cannot be used to determine the probable reduction in lateral capacity. For concrete shear walls, concrete crushing is the primary indication of severe flexural response, and crushing can result from diagonal compression strut action associated with high shear stresses. Major cracks (greater than 0.25 in.) usually indicate shear deformation mechanisms. Each observed damage can be assessed to ascertain if the

ultimate capacity of the wall has been reached. From the experimental evidence, this damage must be severe in order to have reached the ultimate structural capacity of the element. For major crushing and large cracks the steel reinforcement should be inspected to evaluate whether it has been fractured or buckled.] ←

It must be remembered that the ACI 318 code equations for nominal shear strength includes both concrete compression strength and reinforcement ratio terms. It was shown that for slender walls, the lower nominal capacity of the wall calculated using the ACI 318 equations for nominal flexural capacity and nominal shear capacity provide a reasonable prediction of the actual member capacity even though a comparison of the two calculated values does not adequately predict the failure mechanism. For low-rise walls it was shown that their shear capacity for lightly reinforced walls is independent of the reinforcement ratio and that the vertical reinforcement ratio becomes more important as shear friction dominates the wall behavior. All of these equations for nominal strength include terms using the concrete compression strength even though the calculated capacities occur at large deformations when the member has major cracks. Thus, the exclusion of cracked concrete from these equations will give the wrong capacities. Therefore, using crack sizes for the calculation of capacity reduction will lead to erroneous conclusions regarding the loss in load capacity of the damaged members.

The following conclusions are based on the material provided in this paper.

1. Assuming that the walls have reinforcement which exceeds some minimum amount, all reinforced concrete walls have ultimate load capacities greater than those predicted using ACI 31-89 equations. These ultimate capacities occur after significantly large cracks develop in the member or after the concrete crushes.
2. Reduction in lateral load capacity in concrete walls caused by earthquakes requires significant crushing of the concrete and/or fracture of the reinforcement. Wall cracks only indicate that the deformations were large enough to let the reinforcement work.
3. Walls are capable of sustaining repeated cyclic large deformations without significant loss in load capacity. The effective stiffness of these walls decrease with increases in displacement. The importance of this change in stiffness must be evaluated for each specific building.
4. The nominal capacity of slender walls can be reasonably estimated by using the lower of the calculated flexural and shear capacities in accordance with ACI 318-89. The nominal capacity for low-rise walls should use $6\sqrt{f'_c}$ as a lower bound for walls with reinforcement ratios less than 0.38 percent, and the shear friction capacity to a maximum capacity of $10\sqrt{f'_c}$. Small wall cracks do not change these capacities.
5. The selected repair technique should consider the purposes of the repair. A repair to predisaster condition should not be interpreted to imply that it is necessary to restore

stiffnesses to the damaged elements of the building, unless the existing stiffness distribution of the repaired building becomes unbalanced. Similarly, repair to current codes and standards should not be interpreted that walls with minor damage and minimum reinforcement of 0.20 percent should be removed and replaced with new walls with 0.25 percent reinforcement.

6. On the other hand, in some cases increased building stiffness is desired and when an opportunity to economically enhance the performance characteristics of the building exists, these opportunities should be explored. In repairing damaged buildings, it should be remembered that building strength and stiffness are two separate properties under the control of the designer in selecting the most appropriate repair scheme.

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ATTACHMENT C

With Comments

Hassan Sami
1/14/97

Shear Failure in Reinforced Concrete Walls

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ABSTRACT

Following the January 17, 1994 Northridge Earthquake, numerous reinforced concrete and masonry shear walls experienced cracks of different types and sizes. The damaged walls spread over a large geographical area within southern California.

Concrete and masonry shear walls are designed to resist earthquake shaking, based on the assumption that they possess ductility and are capable of inelastic deformation in the reinforcing steel. For this reason the required shear strength must exceed the required flexural strength so that any inelastic deformation associated with loss of stiffness and strength, which might lead to failure, will not occur.

Despite the increased awareness and extensive research in this field, there is still a wide gap between theory and application in design, code requirement, construction, and quality control.

The shear failure in many concrete and masonry shear walls lead to the investigation of how to measure the loss of capacity of these failed walls using the UBC design formulae

SHEAR AND DIAGONAL TENSION

Tension stresses, which are of particular concern in view of the low tensile strength of concrete, of various inclinations and magnitudes, which result from pure shear or the combined action of normal and shear forces may exist in any part of any concrete element. This can impair the integrity of the element if not adequately provided for in the design. It is for this reason that the inclined tension stresses, known as diagonal tension, must be carefully considered in reinforced concrete design and repair.

Diagonal tension cracks form at an average, or nominal, $\text{stress } v_w = V_u/bd = 3.5\sqrt{f'_c}$, where V_u is that shear force at which the formation of the crack was observed. It is evident that at the instant a diagonal tension crack develops, the average shear stress is larger than that given by $v = V/bd$. This is because the pre-existing tension crack has reduced the area of the uncracked concrete, available to resist shear. The amount of this reduction will vary, depending on the length and width of the pre-existing crack. Tests have shown that in a pre-existing cracked section the nominal shear stress at which diagonal tension cracks form and propagate is given by $1.9\sqrt{f'_c}$.

In the early stages of reinforced concrete design, diagonal cracking was considered to be undesirable. However, it is now recognized that diagonal cracking under service load condition is acceptable, provided crack widths remain within the same limits accepted for flexure.

The total shear force V_u is resisted by the combination of:

- 1) a shear force across the compression zone, V_c ;
- 2) a dowel force transmitted across the crack by the "flexural" reinforcement, V_d ;
- 3) the horizontal component of inclined shearing stresses transmitted across the inclined crack by means of aggregate interlock, V_i ; and,
- 4) the reinforcing steel in the direction of the shear force, V_s .

The total resisting shear force is represented by the relationship $V = V_c + V_d + V_s + V_t$.

When shear displacement along an inclined crack occurs, a portion of the shear will be transferred by means of the dowel action of the "flexural" reinforcement. Where the reinforcement bear against the cover concrete, the dowel capacity will be limited by the tensile strength of concrete. Once a splitting crack occurs, the stiffness, and hence the effectiveness, of the dowel action is greatly reduced. This splitting also adversely affects the bond performance of the reinforcement. In a section with reinforcement the contribution of dowel action does not exceed 25% of the total section resistance, and 50% to 70% of the force is resisted by interlock mechanism.

Taylor examined the compression zones of the concrete above diagonal cracks and found out that the shear carried in this area V_c increased slowly to a maximum of 20% to 40% of the total shear force across the section as the sections approached failure. The remainder of the shear must therefore be carried, as mentioned above, by the cracked section, after the breakdown of the aggregate interlock and the dowel action, the compression zone becomes incapable of carrying the increased shear, and the section then fails.

UBC METHOD OF EVALUATING SHEAR STRENGTH

The shear strength of a concrete section is the total of the strength of concrete V_{nc} and the strength of the reinforcing steel V_{ns} . V_{ns} relies on the presence of concrete to develop compression or friction to balance the steel tension resulting from shearing forces induced by flexure or by lateral forces due to earthquakes.

The following is a proposed guideline on how to evaluate the capacity of concrete shear walls:

UBC formula (25-7):

$$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_n f_y)$$

Where $V_{nc} = 2A_{cv}\sqrt{f'_c}$ For $h_w/l_w > 2$

$$V_{nc} = 3A_{cv}\sqrt{f'_c} \text{ For } h_w/l_w = 1.5, V_{ns} = A_{cv}\rho_n f_y$$

Nominal Design Shear:

1. Uncracked section:
 $V_n = V_{nc} + V_{ns}$
2. Cracked section: $1/16" < 3/16"$
 $V_n = 0.5 V_{nc} + V_{ns}$
3. Cracked section: $3/16" < 1/2"$
 $V_n = 0.2 V_{nc} + V_{ns}$
4. Cracked section: $> 1/2"$
 $V_n = V_{ns}$

The above criteria can be used if no analysis or testing is performed on the shear walls.

The following method of testing is recommended.

1. Core A 4" sample of the cracked section.
2. Core A 4" sample of the uncracked section.
3. Core A 4" sample of an epoxy injected section.
4. Compare the shear tests results for different cracks widths to measure the loss of shear capacity.

POST-EARTHQUAKE REPAIR AND UPGRADE OF REINFORCED CONCRETE BUILDINGS

Engineers are often able to utilize their experience to propose repair and/or strengthening methods and schemes for structural damages caused by earthquakes. Recent laboratory test data may be utilized in conjunction with field experience and observations in developing the most effective hazard mitigation programs.

2

Methods of determining the condition of existing structures and material properties need to be identified from as built drawings and field construction data, in the absence of this data, coring of concrete to determine its strength, x-ray and Pathometer equipment can be used to verify the location of reinforcing steel in certain circumstances.

The following are some acceptable methods of repair:

1. Epoxy Resin and epoxy Mortar which possess high tensile strength, higher compressive strength, and greater shear capacity, but low modules of elasticity (approx. 10% of concrete). For most epoxies strength and stiffness decreases linearly from values at about 70° F to Zero at about 400° F. Therefore to maintain required fire endurance, epoxy-repaired elements must be fire proofed when repair and strengthening are required. Epoxy will protect reinforcement from oxidization and corrosion, it has been shown by compression testing of epoxied sections that only 70% to 80% of the existing strength is recovered.
2. Shotcrete, known as "gunite", if the structural engineer determines that epoxy injection will not bring the element or the structure to its pre-disaster strength, stiffness and deformation capacity, the engineer might consider either reconstructing a new element or apply a pneumatic concrete layer of 4" to 6" thick.
3. "Carbon" fiber can effectively restrain and prevent shear or flexural cracks, and add ductility and confinement to the damaged element.
4. In a severely damaged reinforced concrete member, additional steel and concrete can be added to the existing walls by properly anchoring to the existing section to strengthen its strength carrying capacity.
5. If a code trigger mandates upgrading the damaged element or wall, a through analysis of the structural system must be considered.

CONCLUSION

The analysis and recommendation proposed are based mainly on engineering judgement, limited testing on damaged walls at Santa Monica Community College and reflect only the opinion of the authors and not the agencies or organizational affiliation they may have.

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The Effect of Yielding in Reinforcing Steel on Concrete Shear Capacity

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Introduction:

In response to the President's Forum questionnaires on the size of cracks relative to reinforcement yield, the Authors are proposing a simple analysis based on the applications of the fundamental assumptions and practices of Mechanical Behavior of Materials.

Properties of Concrete & Reinforcing Steel:

The properties of these two important dissimilar materials, when used together, give the strength and ductility needed for proper response to all types of loading conditions.

Reinforced concrete elements are subject to different stresses; flexural, torsional, normal, shear, creep, and thermal effect. Any combined effect of these stresses can be reduced to three principal acting stresses in three different directions (x,y,z).

The strength of reinforced concrete under the combined stresses phenomena is not a complete comprehensive science and is based mainly on results gathered from experimental work. For the most effective reinforcing action, steel and concrete must deform together. This can be achieved by the following criteria:

1. A strong bond by chemical adhesion of concrete and the roughness of the steel surface which provides an interlock mechanism.
2. The thermal coefficients of the two materials are "fairly" close which minimizes cracking due to change in temperature.
3. The confinement of steel which possesses high thermal conductivity, and concrete with low thermal conductivity. This limits damage that may be caused by fire to the concrete surface. Hence, the strength of the reinforced concrete element is maintained.
4. The confinement of steel with concrete, will lend itself to protect steel from corrosion.

Strength of Reinforced Concrete Members:

While concrete is best used to resist compression stresses, its tension strength gives rise to structural members resistance to shear and torsional stresses. Steel, on the otherhand, is a high strength material in compression as well as in tension with a usual strength approximately 10 times that of concrete. Steel modulus of elasticity is accepted at 29×10^6 psi, while that of concrete depends on its compressive strength and is estimated at: $57,500 \sqrt{f'_c}$, ($E_c = 3.64 \times 10^6$ psi for $f'_c = 4000$ psi and 4.45×10^6 psi for $f'_c = 6000$ psi).

Based on the assumption that the strain in a well confined reinforcing steel bar is the same as that of the concrete, hence, as one material deforms the other must follow (due to the adhesion and

X

interlock mechanism as mentioned above). The second assumption is that concrete, prior to cracking, does resist tension stresses of small magnitude which contributes to concrete shear strength as expressed by the UBC formula; $V_n = A_{cv} (\alpha_c \sqrt{f'_c} + \rho_n f_y)$.

In the above formulation the two most important parameters are the compressive strength of concrete (reached at 0.002 to 0.003 strain), which is much smaller than the steel can sustain by itself (steel reaches its ultimate strength, 75 ksi for grade 40 steel and 100 ksi for grade 60 steel, at strain of about 0.1), and the second parameter is the yield stress of steel.

As an illustration consider $f_y = 40$ ksi, the yield strain just before yielding occurs can be estimated from $\epsilon_s = f_y / E_s = 40/29 \times 10^3 \sim 0.0018 < 0.002$, which means that concrete has not reached its ultimate strength.

Evaluation of Elongations of Reinforcing Steel:

The following is a table for various lengths of walls and piers. It would be fair to assume that if the crack exceeds these sizes, that the reinforcing steel bars have yielded and that the design shear capacity of the element has been reduced to a value less than $A_{cv} \rho_n f_y$ ($V_n < A_{cv} \rho_n f_y$).

Pier or wall length	4' - 0"	8' - 0"	10' - 0"	20' - 0"
Grade 60 Steel	0.1" > 3/32"	0.2" > 3/16"	0.25" = 1/4"	0.5" = 1/2"
Grade 40 Steel	0.07" > 1/16"	0.13" > 2/16"	0.17" > 3/16"	0.33" > 5/16"

Conclusion:

In the above brief discussion we were able to establish a criteria to see the effect of concrete cracking and steel yielding on the behavior of reinforced concrete shear wall capacity, this criteria is yet to be verified by testing and more rigorous analysis. Mattock and Hawkins, ("Shear Transfer in Reinforced Concrete", PCI, V. 17, No. 2, Mar-Apr. 1972, pp 55-75) have shown that initially cracked concrete sections have less shear capacity than uncracked sections, with low values of $\rho_n f_y$, at about 100 psi, the reduced capacity is approximately in the neighborhood of 40% of the uncracked section subjected to the same shear load.

In-situ strength prior to repair.

TABLE NO. 3-A
EVALUATION CRITERIA FOR DAMAGED SHEAR WALLS

MATERIAL	DETAIL OF SYSTEM	TYPE OF DAMAGE	EVALUATION CRITERIA
Sassi-Ranous Crack Criterion	Reinforced Concrete and Reinforced Masonry Wall with no openings, or any pier within a wall with openings where pier width is not less than twice the story height.	Crack 0.0625 $0" < \text{width} < 1/16"$	No reduction in capacity. Wall must be restored to its pre-event condition considering fire rating of the existing construction.
		Crack $1/16" \leq \text{width} < 3/16"$	The contribution of the concrete to the capacity of the wall shall be reduced by 50%. Consideration must be given to whether or not steel has yielded.
		Crack $3/16" \leq \text{width} < 1/2"$	The contribution of the concrete to the capacity of the wall shall be reduced by 80%. If the reinforcing, across the crack has yielded, the effective length of the wall or pier shall be reduced by the horizontal projection of the crack zone plus two times the wall thickness.
		Crack width $\geq 1/2"$	Capacity shall be the capacity of the reinforcing steel considering permanent elongation of the bars. If the reinforcing, across the crack has yielded, the effective length of the wall or pier shall be reduced by the horizontal projection of the crack zone plus two times the wall thickness.
		Crushing of the concrete at the ends of the wall or pier to include spalling of the concrete exposing the vertical reinforcing.	The effective length of the wall or pier for overturning consideration shall be reduced by the distance from the end of the wall or pier through the width of the crushed zone plus the distance to the closest undamaged vertical bar.
	Shear Wall with openings where pier widths are less than twice the story height.	Crack $0" < \text{width} < 1/16"$	No reduction in capacity. Pier must be restored to its pre-event condition considering fire rating of the existing construction.
		Crack $1/16" \leq \text{width} < 3/16"$	The contribution of the concrete to the capacity of the pier shall be reduced by 50%. Consideration must be given to whether or not steel has yielded.
		Crack $3/16" \leq \text{width} < 1/2"$	The contribution of the concrete to the capacity of the pier shall be reduced by 80%. If the reinforcing, across the crack has yielded, the pier shall be dropped from the system.
		Crack width $\geq 1/2"$	The pier shall be considered as having no capacity and dropped from the system.
		Crushing of the concrete at the ends of the pier to include spalling of the concrete exposing the vertical reinforcing.	
	Spandrel beams	Crack $0" < \text{width} < 1/16"$	No reduction in load delivered to the pier. Spandrel beam must be restored to its pre-event condition considering fire rating of the existing construction.
		Crack $1/16" \leq \text{width} < 3/16"$	The concrete contribution to the capacity of the spandrel to deliver load to the pier shall be reduced by 50%. Capacity of the pier shall be the lesser of the capacity of the pier as determined above, or the capacity of the spandrel to deliver the load.
		Crack $3/16" \leq \text{width} < 1/2"$	The concrete contribution to the capacity of the spandrel to deliver load to the pier shall be reduced by 80%. Capacity of the pier shall be the lesser of the capacity of the pier as determined above, or the capacity of the spandrel to deliver the load.
		Crack width $\geq 1/2"$	The spandrel shall be considered as having no capacity and shall be dropped from the system. Adjacent piers shall be re-evaluated based on a larger height-to-width ratio.

Shear Strength for Uncracked and Cracked Shear Walls

TWO CASE STUDIES ON THE LOSS OF CAPACITY OF CONCRETE SHEAR WALLS (BASED ON THE 1994UBC)

Case 1: Reinforced Concrete Masonry Wall; CMU

Consider an 8" solid grouted R.C.M. Wall as shown in the figure below, with the following properties and geometry:

$$f'_m = 1500 \text{ psi}, f_y = 40000 \text{ psi}, L_w = 120", h_w = 144".$$

Ignore wall own weight, consider the lateral shear to be imposed by boundary elements.

Use #4 @ 24" O.C. EW.

Calculations:

a. Before cracking:

Shear strength is given by UBC Section 2108.2.5.5

$$V_n = V_m + V_s \quad (8-45)$$

Where:

$$V_m = C_d A_{mv} \sqrt{f'_m} \quad (8-46)$$

$$V_s = A_{mv} \rho_n f_y \quad (8-47)$$

For $M/V_d > 1.00$, $C_d = 1.2$ from table 21-K

Therefore;

$$V_m = 1.2 \sqrt{1500} \times (7.63 \times 120) = 42.55 \text{ kips}$$

$$V_s = A_s f_y (d/s) = 0.20 \times (40) \times (116/24) = 38.67 \text{ kips}$$

Hence;

$$V_n = 81.22 \text{ kips}$$

b. After cracking:

Assume a crack 4 feet long along the horizontal projection with a crack width,

across the thickness of the wall, $C_w = 3/16"$.

From the chart or the table this requires a reduction of 50% in the concrete contribution to the shear strength of the cracked wall.

For the 4' cracked section:

$$V_m = 1.2 \times 0.5 \times \sqrt{1500} \times (7.63 \times 48) = 8.51 \text{ kips}$$

For the 6' uncracked section:

$$V_m = 1.2 \times \sqrt{1500} \times (7.63 \times 72) = 25.53 \text{ kips}$$

Total shear resistance of the cracked wall is now = 34.04 kips

Assuming that the reinforcing steel did not yield, no loss of steel shear capacity;

$V_s = 38.67$ kips, as before;

The nominal shear strength of the cracked wall, $V_n = 34.04 + 38.67 = 72.71$ kips.

The loss in shear capacity is; $(81.22 - 72.71)/81.22 = 10.50 \%$.

Case 2: Reinforced Concrete Shear wall:

Consider an 8" R. C. Wall as shown above, with the following properties:

$f'_c = 3000$ psi, $f_y = 40,000$ psi, $L_w = 120"$, $h_w = 144"$.

Use # 4 @ 24" O.C. EW.

Calculatios:

The nominal shear strength is given by the following equation UBC Section 1921.6.4.

$$V_n = A_{cv} (\alpha_c \sqrt{f'_c} + \rho_n f_y) \quad (21-7)$$

Where:

α_c varies linearly from 3 for $h_w/L_w = 1.5$ to 2 for $h_w/L_w > 2.0$

Since $h_w/L_w = 144/120 = 1.2 < 1.5$; $\alpha_c = 3$

a. Before cracking:

$$V_c = \alpha_c A_{cv} \sqrt{f_c} = 3 \sqrt{3000} \times (8 \times 120) = 157.74 \text{ kips;}$$

$$V_s = A_s f_y (d/s) = 0.2 \times (40) \times (116/24) = 38.67 \text{ kips;}$$

$$\text{From equation (21-7), } V_n = V_c + V_s = 157.74 + 38.67 = 196.41 \text{ kips.}$$

This is the capacity of the shear wall before cracking.

b. After cracking:

Assume a crack 4' along the horizontal projection with a crack width $C_w = 3/16"$.
From the graph or the table this will require a reduction of 50% in the concrete contribution.

For the 4' cracked section:

$$V_c = 0.5 \times 3 \times \sqrt{f_c} \times (8 \times 48) = 31.55 \text{ kips;}$$

For the 6' uncracked section:

$$V_c = 3 \times \sqrt{f_c} \times (8 \times 72) = 94.65 \text{ kips;}$$

$$\text{The total concrete resistance is } = 126.20 \text{ kips;}$$

Assuming that the reinforcing steel did not yield, i.e. no loss in steel shear capacity;

$$V_s = 38.67 \text{ kips;}$$

The nominal shear strength of the cracked wall, $V_n = 126.20 + 38.67 = 164.87 \text{ kips}$

The loss in shear capacity is; $(196.41 - 164.87) / 196.41 = 16\%$

This % will be more than double if 100% loss of concrete is assumed.

ATTACHMENT D

Steel Moment Frame

- ▶ A36/A572 versus dual-cert. (strong column, weak beam)
- ▶ Welding quality control (incorrect procedure, back-up bars)
- ▶ K-line notice from AISC (WF root area non-ductile)
- ▶ Welding electrode/process (FCAW-SS vs .SMAW)
- ▶ FEMA 267 has inspection, repair guidelines for Zone 4 constructions
 - EQ Magnitude ≥ 6.5 , 0.2g ground motion
 - Any EQ, if ground motion 0.3g or greater
 - For buildings in other seismic zones, reduce by $A_a/0.4$ but need not reduce to less than 0.15g

W E L D I N G

1. Unless noted otherwise, all new welds shall be made per welding note 6. All shielded metal arc welding (SMAW) electrodes shall be of the low hydrogen type and shall be subject to the requirements of Section 4.5 of AWS D1.1-94. For SMAW, the maximum permitted electrode diameter shall be per welding note 6 Reference Section 4.6.3(4) of AWS D1.1-94.
2. All inspectors are to be AWS Q.C.-1 certified in accordance with Section 6.1.3.1. in AWS D1.1-94.
3. A written welding structural procedure specifications (WPS) shall be developed by the fabricator and submitted to the engineer for approval prior to beginning welding operations. The WPS shall include the welding parameters recommended by the electrode manufacturer. See AWS D1.1-94, Appendix E, Form E-1. Joint details to suit existing conditions. The welding electrode manufacturer's specifications shall be attached to the WPS. All welders and inspectors shall be informed of the WPS and shall retain a copy. The WPS shall include the following:
 - A. The written WPS shall contain all the necessary information required by the code, this specification, and any other information necessary to produce welds that are in compliance with these requirements.
 - B. The WPS shall list the applicable base metal types and thickness.
 - C. The WPS shall list the welding joint details, including joint type, weld type, joint geometry, and applicable dimensions. Individual weld passes shall be identified in the sketch and numbered to identify the sequence of their deposition. The sketch shall identify the maximum layer thickness and bead widths. In no case shall the layer thickness exceed 1/4", nor shall the maximum bead width exceed 5/8".
 - D. The WPS shall list the applicable welding process. Weld process (SMAW) to be submitted to city building department for approval prior to start of construction.
 - E. The WPS shall indicate the position of welding.
 - F. The WPS shall list the filler metal per AWS D1.1-94 and electrode specification and classification, as well as details regarding shielding material used.

- G. The WPS shall indicate the minimum preheat requirements and interpass temperatures. The minimum specified preheat shall meet the requirements of AWS D1.1, Table 4.3 and AISC Section J2-7 requirements for jumbo sections. The WPS shall also indicate post heat requirements, if required or not.
 - H. The WPS shall list all applicable electrical characteristics for the process employed. The WPS shall clearly indicate the specific values required for each welding pass. These electrical characteristics shall include at a minimum the following:
 - 1. Electrode diameter.
 - 2. Type of current and acceptable ranges of current measured in amperage. For wire feed processes, indicate manufacture recommended wire feed speed, melt off rate and deposition rate.
 - 3. Voltage (for all processes)
 - 4. Actual field condition travel speed & manufacture data for travel speed.
 - 5. electrode extension (stick out) for wire feed processes.
 - 6. Amperage, voltage and electrode extension (as applicable) shall be within the filler metal manufacturer's recommendations (compare to D1.1:4.6.2).
 - I. The diameter of the electrodes specified on the WPS shall not exceed the pre-qualified limits shown in AWS D1.1. paragraphs 4.6.3, 4.7.3, 4.14.1.2 and 4.14.1.3, as applicable. In addition, in the case of FCAW, the maximum electrode diameter shall not exceed 1/8".
 - J. All electrodes listed in the WPS shall be low hydrogen types.
 - K. A copy of electrode manufacturer's technical information and certificate of conformance shall be attached to the WPS.
4. A pre-construction meeting between the engineer of record, the fabricator, the erector, the contractor and the inspectors shall take place to discuss the welding procedure specification (WPS) to insure the quality of work that the engineer of record is trying to establish. The welding electrode manufacturers'

specifications should be attached to the WPS on all jobs. All welders and inspectors shall be informed of and must adhere to the WPS and shall retain a copy.

5. To assure the proper amperage and voltage of the welding process, the use of a hand held calibrated amp and volt meter shall be used. This equipment shall be used by the fabricator, erector and the inspectors. Amperage shall be measured at the lead and voltage shall be measured at the arc with this equipment. Travel speed and electrode stick out shall be verified to be in compliance with the electrode manufacturer's recommendations.

6. Summary of typical requirements for prequalified SMAW WPS's:

Variable	Position	Weld Type	Size	Code paragraph
Maximum Electrode Diameter	Flat	All Except Root Passes	5/16"	4.6.3 (1)
	Horizontal	Filets	1/4"	4.6.3 (2)
		Groove	3/16"	4.6.3 (5)
	Vertical	All	5/32"	4.6.3 (5)
	Overhead			4.6.3 (4)
Maximum Root Pass Thickness	All	Groove	1/4"	4.6.5
Maximum Single Pass Fillet and Maximum Weld Root Pass Thickness	Flat	-	3/8"	4.6.6 (1)
	Horizontal	-	5/16"	4.6.6 (2)
	Vertical	-	1/2"	4.6.6 (3)
	Overhead	-	5/16"	4.6.6 (2)
Maximum Layer Thickness	Flat	All	1/8"	4.6.7 (1)
	Horizontal	All	3/16"	4.6.7 (2)
	Vertical	All	3/16"	
	Overhead	All	3/16"	

7. Summary of typical requirements for prequalified FCAW WPS's:

Variable	Position	Weld Type	Size	Code paragraph
Maximum Electrode Diameter	Flat	All	1/8"	4.14.1.2
	Horizontal		1/8"	
	Vertical		3/32"	
	Overhead		5/64"	
Maximum Root	All	Groove	1/4"	4.14.1.5

Welding.txt

Pass Thickness
(except root &
surface layers)

Maximum Single Pass Weld	Flat Horizontal Vertical Overhead	Fillet	1/2" 3/8" 1/2" 5/16"	4.14.1.3
Maximum Single Pass Layer Width, Root Open'g is 1/2" or Greater	All	All	Use Split Layer	4.14.1.5
Maximum Single Pass Layer Width	Flat Horizontal Overhead Vertical	Groove	1/2"	4.14.1.5

8. Welding contractor shall protect all existing building elements against fire or damage and shall provide a full time fire watch equipped with fire extinguisher during all cutting and welding operations. In addition, welding techniques shall be used to minimize potential fire damage.
9. Removal of backup bar and damaged material can be done by grinding or air arc gouging. Areas backgouged by air arc to be cleaned up by grinding. Surfaces on which weld metal will be deposited shall be smooth, uniform, and free from fins, tears, fractures, and other discontinuities which would adversely affect the quality or strength of the weld, per AWS D1.1-94 Section 3.2.1.
10. Welding to be done by welders certified by AWS, and the Los Angeles City Building Department for structural steel.
11. All repair welds to be complete joint penetration welds unless specifically noted otherwise.
12. Individual welds shall be carried continuously to completion before joint is allowed to cool below the minimum specified preheat and interpass temperature.
13. Beam welding sequence: When welding is required at both ends of a beam, one end shall be allowed to cool before welding begins at the opposite end.
14. Preheating is required on all tack welds not incorporated into final welds. See AWS D1.1-94, Section 3.3.7.

15. 100% ultrasonic weld testing by an ASNT level II inspector, supervised by a level III inspector, is required for all complete joint penetration welds.
16. Travel speed and electrode stick out shall be verified to be in compliance with the electrode manufacturer's recommendations.
17. All surfaces shall be cleaned free of rust, paint, debris, etc. prior to welding.
18. When a FCAW process is used, the welding procedure specification shall conform to welding note 7 and the electrode manufacturer's recommendations. Note a FCAW process shall not be used without prior written approval by the structural engineer. Note that filler metal shall have a notch toughness of 30 ft/lbs average (20 ft/lbs minimum) at zero degrees FAHRENHEIT
19. All welds shall be terminated at the end of a joint in a manner that will ensure sound welds. Whenever necessary this shall be done by use of extension bars and run off tabs. No weld dams are allowed. Run off tabs used at beam flange connection shall be removed and the ends of the weld shall be made smooth and flush with the edges of abutting parts per 3.12.3 of AWS D1.1-94.
- 20.A A complete joint penetration groove welds shall have a filler metal with a notch toughness of 30 ft/lbs average (20 ft/lbs minimum) at zero degrees Fahrenheit.
- 20.B Column flange replacement plates and beam flange splice plates shall have a notch toughness of 20 ft/lbs average at 70 degrees Fahrenheit. Also, rolling direction of grain shall be indicated on the plate and installed so that grain direction is parallel to longitudinal axis of the member.
21. Requirements for complete joint penetration groove welds.
 - A. The root pass shall begin at the end of the beam flange. After the arc is initiated, travel shall progress continuously from one edge of the beam flange to the other and the weld shall be terminated at the weld run off tab.
 - B. The half length root pass shall be thoroughly cleaned.
 - C. The start of the weld in the weld access hole area shall be visually inspected to ensure fusion, soundness, freedom from slag inclusions and excessive porosity. The resulting bead profile shall be

suitable for obtaining good fusion by the subsequent pass to be initiated on the opposite side of the beam web. If the profile is not conducive to good fusion, the start of the first root pass shall be ground, gouged, chipped, or otherwise prepared to ensure adequate fusion.

- D. The second half of the weld joint shall have the root pass applied before any other weld passes are performed. The arc shall be initiated in the area of the start of the first root pass, and travel shall progress to the end of the joint, terminating on the weld run off tab.
- E. Each weld layer shall be completed on both sides of the joint before a new layer is deposited.
- F. Each weld layer shall be completed on both sides of the joint before a new layer is deposited.
- F. Individual welds shall be carried continuously to completion before joint is allowed to cool below the minimum specified preheat and interpass temperature.
- G. Deviation from the preceding procedures may be made, providing the contractor submits in writing an alternate sequence that is approved by the structural engineer prior to fabrication.

22. Requirements regarding preheat

- A. Preheat the entire joint to a minimum of 225 degrees F before performing any work such as removal of backup bars, modifying cope holes, tack welding, etc.
- B. Preheat the entire joint to a minimum of 350 degrees F. Preheat temperature shall be measured at a distance of 3" in all directions away from the joint, and shall extend through the column flange thickness over that same area. Preheat shall be verified by an inspector before welding proceeds.
- C. All preheating shall be done uniformly. Where possible, provide heating from the back side (root side) of the member. Members should always be brought to temperature slowly.
- D. For group 4 & 5 shapes, preheat to 225 degrees F prior to thermal cutting of access holes. For all thermal cutting of access holes, inspect thermally cut edges using magnetic particle or dye penetration methods.

- E. Maximum interpass temperature 550 degrees F to be monitored on column flange maintain preheat temperature when welding is interrupted.

ATTACHMENT E

HAZARDOUS MATERIAL RELEASES ASSOCIATED WITH EARTHQUAKES

BUILDING STRUCTURAL FAILURES

This section discusses the types of failures that have been directly responsible for hazardous materials releases in earthquakes. However, general system failures or accidents that are not directly caused by an earthquake may also occur. For example, damage to a cooling system or interruption of a power supply can cause secondary problems, such as spills or fires, and damage to adjacent facilities may lead to problems in a plant.

Types of Failures

The failures that have occurred can be grouped into eight different categories.

Building structural failure can lead to hazardous materials releases by collapse of the structure onto areas containing hazardous materials, including pipelines and shelving. For example, the failure of the unreinforced masonry dry cleaners in the Coalinga Earthquake resulted in a minor spill of cleaning agents (conv. with Fredrickson at Coalinga Fire Dept., 1989). A six-story tower used for storing chemicals was damaged and resulted in release of chemicals at the Unisola Soap Plant in the San Salvador Earthquake (Ledbetter and Bommer, 1987).

Excessive flexibility of a structure can also lead to problems. Even in well-designed structures, upper floors are more susceptible to hazardous materials problems than lower floors. For example, at the Univ. of California Medical Center in San Francisco, all of the spills occurred above the ninth floor (conv. with Reich and Lee at UCSF Environmental Health, 1990).

Pounding of adjacent structures also can lead to damage to contents, particularly at the level equal to the height of the lower building.

Hazardous materials releases in buildings which also have major structural failures can complicate and delay building demolition. For example, a biological spill in the basement of the Olive View Hospital during the San Fernando Earthquake had to be cleaned up before the building could be torn down (conv. with Anderson at CSULA, 1989). In that same earthquake, the California Office of Emergency Services had to use radiological detectors to determine if a spill of radioactive materials used for medical purposes had occurred before the Veterans Administration Hospital could be demolished (conv. with Reitherman, 1990). Asbestos may also need to be cleaned up.

ASBESTOS PROBLEMS

Asbestos contamination is a poorly-publicized form of hazardous materials release that can result from earthquake damage. ABAG research identified over 80 buildings or facilities which had asbestos problems as a result of the Loma Prieta Earthquake.

Following an earthquake in which asbestos is released, there is a higher than normal potential for building occupants, clean-up, or renovation personnel to be exposed to increased levels of asbestos. Claims for personal injury may be filed, further complicating recovery. Specialized mandatory asbestos removal and abatement procedures can result in very expensive clean-up costs. Unplanned temporary loss of use of buildings can last for *months*.

The most common source of contamination is the asbestos in sprayed-on acoustic or fireproofing insulation shaking loose and falling. Asbestos that has fallen on suspended ceiling tiles can create a hazard.

Another major cause of asbestos contamination is the rubbing loose of insulation on pipes due to contact with the pipe supports, other pipes, or a building structural member.

Asbestos may need to be cleaned up prior to building demolition, delaying and complicating that problem. For example, asbestos needed to be removed from the damaged May Company facility in Whittier before it could be demolished (conv. with Morkus at City of Commerce, 1989).

Clean-up and renovation activities may create an even greater release of asbestos than that caused by the earthquake!

PIPELINE BREAKS

Because of their widespread distribution, natural gas leaks are the most common releases during earthquakes and are responsible for many of the resulting explosions and fires. This damage demonstrates the vulnerability of all underground pipelines. They can be broken by soil movement, particularly if there is some pre-existing deterioration in the lines. Such movement can be caused by liquefaction, landsliding, lateral spreading, and other forms of ground failure. Recent underground storage tank regulations tend to promote the identification and containment of leaks in pipelines associated with underground tanks.

Above-ground piping is especially vulnerable to breakage from differential movement between pipes, tanks, and buildings. Typically, the sliding of an unanchored tank or piece of equipment ruptures short spans of connection pipe. Pipelines also may be subject to failure at the ground-building interface and where they cross building expansion joints. Other damage prone areas are where pipes intersect at right angles, and at threaded, rather than welded, joints. Ground failure can trigger damage to tank foundations and pipeline supports. Damage can also occur from

falling pipe supports and impact from other equipment or structures. Even if the pipeline itself contains water, rather than hazardous materials, the loss of that water can impede fire-fighting capability.

Recent earthquakes, including the 1978 Chilean Earthquake, the 1983 Coalinga Earthquake, the 1987 Whittier Narrows Earthquake, and the 1989 Loma Prieta Earthquake, actually demonstrated that well-designed process pipelines can perform well in an earthquake. In those earthquakes, the damage to pipelines was usually limited to sections that were corroded or anchored at two locations experiencing large relative displacement.

TANK FAILURE

Large Cylindrical Storage Tanks

Tank failures seriously affect facility operations and can result in large financial losses.

Large cylindrical storage tanks are vulnerable to several types of failure.

"Elephant's foot" buckling failures at the base of tanks are caused by horizontal forces, and often result in the loss of contents. Such failures are not limited to petroleum tanks, but can occur in tanks containing everything from water to wine.

The sloshing of contents can cause tank roofs to collapse or failure of the upper portions of tank walls.

Sliding is common for unanchored tanks during earthquakes. If there is enough flexibility in the connecting pipelines, such movement can occur without any loss of contents. Storage tanks at industrial facilities did not leak in spite of sliding 4 inches in both the Coalinga and Loma Prieta Earthquakes.

Even anchored tanks can move. For example, the Amarillo Water Co. water tank (60' tall and 20' in diameter) in Whittier leaked only slightly, although there was damage to the anchor bolts. Each of eight 2" diameter steel bolts elongated 3/4" and had to be retightened (Degenkolb Assoc., 1987).

Corrosion at the base of tanks can also be a problem. For example, at a large petroleum refinery in Miyaki-Ken-Oki, Japan in 1978, three of 87 large tanks failed, spilling 19 million gallons of oil, half of which overtopped containment dikes and entered the adjacent bay. This tank failure was attributed to "pitting" corrosion of the tank bottom (Ellingwood, 1980).

Tanks also can settle into gravel pads, pointing to a need for concrete foundation pads.

Tank walls can be damaged due to inadequate detailing at connections with external pipes, valves and ladders.

Leaks can also occur due to the failure of internal baffles.

Gas-filled tanks can also be damaged. During the Miyagi-Ken-Oki Earthquake, at the Sendai City Gas Facility, a large cylindrical propane storage tank (approximately 125 feet in diameter and 89 feet high) containing 500,000 cu. ft. of propane at approximately 10 atmospheres collapsed and burned (Ellingwood, 1980).

In the 1964 Niigata (Japan) Earthquake, conventional cylindrical tanks for petroleum products with floating roofs caught fire from friction between the seal of the floating roof and the edge of the tank. As a result, these floating roofs now have plastic seals (Tokyo Petrochemical Complex, 1981). This practice is not universal in California's oil facilities.

Even taking a tank out of service may not render it harmless! In Hollister, as a result of the Loma Prieta Earthquake, an empty silo fell on an adjacent building, causing extensive damage.

In the 1978 Chilean Earthquake, the major problem was damage to large vertical storage tanks. An average of 10-15% of the tanks failed, often losing their contents (Wyllie, 1986).

Elevated Tanks

Elevated tanks can topple if inadequately supported. Elevated tanks typically perform worse in earthquakes than ground mounted structures. For example, large vertical tanks containing molasses at the Coalinga Feed Lot tilted and in one case overturned. The entire steel frame support structure was badly deformed and pushed out of alignment (Degenkolb, 1983).

Sloshing

Another type of tank "failure" is not a structural failure of the tank, but a problem with open-top tank design. Earthquakes can cause sloshing of various liquids, such as plating solutions, and lead to mixing of incompatible chemicals and materials on the building floor. Such mixing can generate toxic gases, such as hydrogen cyanide. If the secondary containment trenches are shallow, continued sloshing can result in a loss of containment and material escaping to the storm drain or sanitary sewer. Although rarely documented in past earthquakes, ABAG staff have documented sloshing in at least 40 locations in the recent Loma Prieta Earthquake.

Horizontal Pressure Vessels

Pressure vessels can shift and bounce on their supports. Chlorine tanks at a Coalinga treatment plant slid up to 10 inches on standard saddle supports.

Fortunately, there was sufficient slack in the attached tubing to prevent rupturing of the lines.

Pressure vessels can even roll over if inadequately anchored. This occurred during the 1971 San Fernando Earthquake, as well as in the Santa Cruz Mountains as a result of the 1989 Loma Prieta Earthquake.

Gas Cylinders

Gas cylinders are particularly vulnerable to leaks when they are being filled or used. In the Whittier Earthquake, a 1-ton cylinder being filled with chlorine started to roll down the loading platform and broke the connection to the filling hose at the screwed threads. This failure resulted in a significant chlorine leak (conv. with Santa Fe Springs Fire Dept. and company personnel, 1989).

Because they are so tall relative to their diameter, 150-pound gas cylinders are prone to falling over in earthquakes. Therefore, many companies routinely chain their cylinders. This requires *people* to chain them, which sometimes does not happen. In addition, the restraint must be *adequate*. C-clamps routinely come loose in earthquakes. Flimsy chains can break, and chains anchored into gypsum board pull out. *Cylinders can slip out from under single chains.*

Valves

A valve may also be the location for a material leak. Although problems with valves are quite rare, a few problems have occurred. For example, when power was restored following the Loma Prieta Earthquake, a float valve jammed in a business's plating shop, resulting in the overfilling of a tank (conv. with company staff, 1989). In the same earthquake, a small leak of ammonia occurred at a cold storage facility in Watsonville when the packing on a valve failed (conv. with Watsonville Fire Dept. and company staff, 1990). In the Coalinga Earthquake, the valves on a 1" line to an oil clarifying tank shook open, causing a "major" oil spill (Chan and others, EQE, 1986). Finally, in the Saguenay River Earthquake in Quebec, Canada, a pressure relief valve at a smelter failed and vented 8 tons of hydrogen fluoride gas into a building where it seeped to the outside and dissipated (EQE, 1988).

Falling Containers and Shelves

One of the biggest problems for laboratories (at schools, hospitals, and private companies) results from the falling and breaking containers of chemicals (many of which are incompatible) or of the shelving on which they are stored.

Shelves holding chemical bottles can collapse. Even if they do not completely collapse, they can bow, causing the chemical bottles to fall to the ground.

In the Coalinga Earthquake, an acid spill at the high school chemistry lab ate through the floor and dissolved part of the concrete foundation in the basement. It took

three days to clean up that incident at an estimated cost of \$90,000 (conv. with Coalinga School District staff, 1989). Of more concern, however, was the spill which occurred in the nearby west Hills Community College. The spill occurred when strong acids and bases (including sodium hydroxide) left on the lab counters during an overnight experiment fell to the floor and mixed. No one took any pictures of this incident, in large part because of the fear of an imminent explosion. This fear of an explosion was exacerbated because the gymnasium at the facility was being used as an emergency shelter (conv. with Gilmartin at West Hills College, 1989).

Dozens of labs had chemical spills at the Lawrence Livermore National Lab as a result of the Livermore Earthquake. One lab had a small spill from a petri dish with 100 pellets totaling 0.1 curie of radioactive material spill (LLNL Earthquake Impact Analysis Committee, 1980). This incident is one of only a handful of documented spills of small amounts of radioactive materials.

During the Whittier Earthquake, Cal State LA experienced a number of spills. The cabinet doors and drawers flew open, allowing the contents to fall out.

Sometimes, a relatively small incident can lead to hundreds of thousands of dollars of damage. For example, on the 8th floor of the Physical Sciences Building at Cal State LA, a bottle of less than one gallon of kerosene containing sodium metal and located under a fume hood, was knocked over during the shaking of the Whittier Earthquake. A water leak from a 1" line from a safety shower (which had been broken at a threaded connection during the shaking) reacted with the sodium, and the resultant hydrogen gas ignited. The overhead sprinkler system was non-operational, preventing the sprinkler system from putting out the fire. The apparatus grid in the room had mercury manometers attached which were destroyed and spread contamination. Finally, the fire and earthquake combined to create an asbestos problem that had to be cleaned up. The asbestos clean up alone cost \$237,000 (conv. with Lipson and Anderson at CSULA, 1989).

At the same facility a fruit fly experiment being conducted by a genetics class was released from confinement. Insecticides and pesticides were available to prevent the flies from escaping into the environment (conv. with Lipson and Anderson at CSULA, 1989).

Other "biological" spills occurred at atleast one hospital's specimen library in the Whittier Earthquake. The worker who went to clean up the mess was overcome by the formaldehyde fumes and passed out in an adjacent hall (conv. with hospital safety manager, 1989). A spill of similar size and materials occurred at a Bay Area hospital during the Loma Prieta Earthquake. These clean up workers were not affected by the formaldehyde because they were using proper equipment (conv. with hospital environmental safety staff, 1989).

Problems of falling containers and shelves are not limited to laboratories.

Falling containers in retail stores and warehouses also create a problem. At a retail store in Mountain View, containers containing malathion and chlorine fell and mixed in the Loma Prieta Earthquake, resulting in the evacuation of the building (conv. with store staff, 1990).

The main fire in the downtown Coalinga area following the earthquake there occurred at a steak house where vapors from broken liquor bottles were ignited by a nearby gas heater (conv. with Coalinga Fire Dept., 1989).

Even homes contain liquor bottles and cleaning materials. A garage often contains petroleum products, pesticides and other hazardous materials. Pool chemicals are also common. These chemicals often spill and mix in earthquakes.

Sliding and Overturning Industrial Equipment

Tall, narrow objects fall over and broader shapes slide, sometimes causing secondary damage to objects that they hit.

Stoves and other equipment can slide, causing damage to attached gas lines. Water heaters can fall over or tilt, causing their attached gas lines to fail. (However, the water line usually fails first because it connects to the top of the heater, while the gas connects to the base.) Broken gas lines can ignite and cause fires.

If ceiling-mounted equipment is not adequately braced, it can swing and break attached piping. Such a problem with an evaporator at a freezing facility caused a release of 5000 pounds of ammonia in the Loma Prieta Earthquake (conv. with Watsonville Fire Dept. and company personnel, 1990).

There is a common myth that something can be "too heavy" to move. Unanchored 1000 lb. muffle ovens slid from the lab bench tops to the floor in the Physical Sciences Building at Cal State LA (conv. with Lipson and Anderson at CSULA, 1989).

Even anchored equipment is vulnerable to damage. A screw on a Nuclear Magnetic Resonance Imager (NMR) unthreaded in the Loma Prieta Earthquake, allowed the NMR to become misaligned (conv. with company personnel, 1989). Anchors can also break, as occurred on a 12kV transformer during the Coalinga Earthquake (Degenkolb and Assoc., 1983). Even if the anchor holds, this is no guarantee that the equipment internals will not shake apart; one company learned this lesson after 4 days of lost production following a 4.5 M earthquake centered very near their facility (conv. with company staff, 1989).

Sometimes the building contents can be so large and massive that it, rather than the direct shaking, can lead to structural failure. Both tilt-up and light metal warehouses in Hollister were damaged severely (to the point of partial collapse) by the overturning of pallets of tomato cans in the Loma Prieta Earthquake. Interestingly, the tilt-up buildings were being repaired while the light metal building was being torn down (ABAG staff observations, Oct. 1989 and March 1990).

Transportation Accidents

In the 1978 Santa Barbara Earthquake, magnitude 5.1, a train derailed shortly after the earthquake due to damaged tracks. Thirty to 50 cars derailed at 50 mph (Santa Barbara News Press article, 1978). Luckily, none of these cars were carrying hazardous materials, but it demonstrates the potential for a greater incident.

Train cars do not have to be moving to be damaged. An engine was stopped at Pt. Reyes Station when the San Francisco Earthquake occurred, and tipped over. This incident points out what can happen to tank cars of chlorine at sewage treatment plants and industrial facilities.

Damaged roads and overpasses could certainly cause accidents involving transported hazardous materials. One can easily imagine the additional damage that would have been caused if the Cypress structure collapse in Oakland due to the Loma Prieta Earthquake had crushed a truck, causing a toxic gas release. As it was, there were five fires which resulted from that collapsed structure. Three of the fires were caused by the ignition of gasoline when vehicles were crushed on the lower deck. A fourth fire was caused by the ignition of diesel oil when a vehicle was crushed on the lower deck. The fifth fire was caused by the ignition of gasoline when a vehicle fell off the upper deck onto the surface street below the freeway (conv. with California Department of Transportation, 1989).

There were no fires resulting from the failure of the Bay Bridge in the Loma Prieta Earthquake. However, this failure, more than the Cypress structure, crippled the transportation system from the East Bay to San Francisco for a month, and resulted in reduced emergency response capabilities. The failure of smaller bridges and overcrossings can also disrupt emergency response, the subject of the following sections on problems.

Special Response Problems

Water supply may be disrupted for both process and fire fighting applications. Excessive water, such as from broken sprinkler lines, also can lead to problems with pyrophoric chemicals and computer-driven process control systems.

Power may be disrupted for several days, rather than just a few hours. Battery back-ups designed to provide power for a few minutes to a few hours may not be adequate or advisable. For example, because of the fear of ignition of natural gas, PG&E did not restore electrical power in some areas for several days following the Loma Prieta Earthquake.

Telephone communications may be disrupted due to overuse or equipment failures (particularly with internal PBX systems). If repeaters on walkie-talkie or radio systems are not on back-up power, these systems can be crippled, as occurred at one Bay Area hospital in the Loma Prieta Earthquake (conv. with environmental safety staff at hospital, 1989). Back-up power for communications can also be inadequate. The battery back-up for the telephone equipment at San Francisco's

City Hall lasted about 10 hours, but the electrical service from PG&E was not restored for several days (conv. with Lew, Seismic Safety Program Manager of San Francisco Dept. of Public Works, 1990).

Several factors can contribute to the emergency staff being less available than planned. Fallen lights and equipment overturn and block corridors. Offices in shambles also disrupt building evacuation. Thus, transferring emergency response staff to emergency locations becomes difficult. Hazmat clean up crews may not have the required **earthquake** emergency identification and may not be allowed past barricades erected to prevent access due to structural problems in a neighborhood. Finally, the emergency response staff can even be injured in the original shaking.

Manual alarms or emergency shut-off systems may not be triggered because the person responsible for pushing the buttons is under a desk. This scenario actually happened to one company in the Loma Prieta Earthquake; that company is now installing shut-off devices triggered by shaking (conv. with company staff, 1989).

The demand for clean-up crews is great, which resulted in some companies paying a premium for services, such as asbestos abatement, following the Loma Prieta Earthquake.

Finally, emergency response facilities themselves may be damaged. In the Loma Prieta Earthquake, the San Mateo County Emergency Operations Center had to be evacuated and moved upstairs in the County Administration Building for about an hour due to a leaking natural gas line. One of two sprinkler line breaks threatened essential communications equipment (conv. with Paxton at San Mateo County, 1990).

Appendix D

Attendance List for REI Training

PROJECT HANFORD MANAGEMENT CONTRACT POST NPH
Emergency Response Training
March 18-21, 1997

Attendee	Org. Code	Phone #	MSIN	Team
T. Conrads	8C400	6-7642	B1-17	D.E./R.R.
DAVID NARE	8R120	3-3856	S3-97	HFD
Don Zimmerman	8R120	3-3320	S3-97	HFD
Dale Harrison	19270	3-3495	L1-08	324/FacRPT
Greg Glover	15510	3-3096	T4-20	PR
Gary Waggonblast		6-5736	S0-04	DET
Paul Izthington	17700	3-2110	S6-21	PR/DET/Purex
VILL FOREMAN	19210	6-9370	L1-01	324/RPT
Don Harts Horn	DT921	6-5406		Seismic Maint
JERRY KEELIN	8R210	6-3345	L7-97	HFD
Robert Russell	15510	3-4355	T4-20	PFP
RICHARD THOMAS	15510	3-5046	T4-20	PFP/PR
Jim Steffen	19100	6-0484	N1-47	300 Area RPT
Jim Rich	19120	6-3890	L6-26	300 Area DET
BRUCE R. BOWMAN	18230	6-0446	N2-56	FTE
LOUIS A. VALLÉ	438	2-3194	G3-08	PHIK
Steve Resdel	DT921	6-9932	K6-81	PNNL
DAVID W. MERTZ	16A00	2-0359	J6-70	BWAL
WILLIAM A. FRISL	2T220	373-3946	X3-74	DGS'S
Rich Giller	0438	376-2662	E6-46	FDNW
WAYNE A. ADANEK	0438	373-4197	G3-17	FDNW
Swint K. Kanyel	0438	372-1244	S0-04	FDNW
C.D. William	17000	373-1587	S5-66	BWAL/PUREX
S.A. Farge	0438	376-6489	E6-46	FDNW
M.A. MAQ	0438	376-6837	G3-17	"

Appendix E

Attendance List for Supplementary DE Training

PROJECT HANFORD MANAGEMENT CONTRACT POST NPH
Emergency Response Training
March 18-21, 1997

[illegible]

Appendix F

Test Questions for Rapid Evaluation Inspectors

Name _____

Rapid Evaluation Inspector - Exam**Circle the most appropriate response**

1. The role of the Post-NPH Technical Representative is to
 - a) direct the actions of the Building Emergency Director/Incident Commander (BED/IC).
 - b) coordinate inspections to locations or facilities where there is no emergency response organization.
 - c) perform detailed evaluations.
 - d) act as spokesperson to media during a post-NPH emergency.

2. After an NPH event, REI inspectors should
 - a) immediately begin to enter nearby buildings and conduct inspections.
 - b) re-enter buildings only with the approval of the IC.
 - c) go home to make sure that families are safe, then contact Patrol Headquarters.
 - d) contact Patrol Headquarters to activate the Emergency Operations Center.

3. The most significant earthquake hazard to Hanford DOE buildings is
 - a) ground liquefaction.
 - b) strong shaking.
 - c) surface rupture due to fault moving.
 - d) ground subsidence due to strong shaking.

4. "Do Not Cross Line" tape is used to:
 - a) barricade local areas near potential falling hazards.
 - b) barricade buildings which are unsafe.
 - c) keep building workers out while the inspection is being conducted.
 - d) Both (a) and (b).

5. A building should be marked as "Unsafe" due to ground fissures or movement:
 - a) if signs of ground movement are evident anywhere near the building.
 - b) if fissures or ground movement have occurred directly beneath the building and significant foundation damage is evident.
 - c) if cracks are observed in any foundation.
 - d) Both (a) and (b).

Rapid Evaluation Inspector - Exam, Page 2

Slide Evaluation. Circle the most appropriate response

Photo No. 1. The building is concrete construction. Observed crack width varies from 0.5 to 1 inch width.

- a) Significant structural damage. Designate "Unsafe."
- b) Limited Entry, however, mark width and monitor for further movement.
- c) Limited structural damage. Designate "Limited Entry."
- d) Cabinet is a risk of toppling. Cordon "Area Unsafe."

Photo No. 2. The building is reinforced masonry construction.

- a) Significant structural damage. Designate "Unsafe."
- b) Designate "Limited Entry." Mark width and monitor for further movement.
- c) Limited structural damage. Designate "Limited Entry."
- d) Designate Unsafe AND cordon area beneath canopy as "Area Unsafe."

Photo No. 3. Within a building this wall is observed. Expanded metal mesh is visible where portions of the wall surface have fallen to the floor.

- a) Wall is non-structural lath-and-plaster partition. Building structure condition cannot be assessed from this damage.
- b) Significant structural concrete damage. Designate "Unsafe."
- c) Limited structural concrete damage. Designate "Limited Entry."
- d) Localized damage at this one location is insufficient to define overall structure safety.

Photo No. 4. Within a building, the anchor bolts at the base of this steel column have sheared. Column is laterally displaced about 2 inches.

- a) Minor damage does not appear to be a concern. Designate "Safe."
- b) Column may be unstable. Designate "Limited Entry."
- c) Column may be unstable. Designate "Unsafe."
- d) Column may be unstable, but overall building may not be affected by this condition. Cordon area to adjacent columns (to all sides) as "Area Unsafe."

Rapid Evaluation Inspector - Exam, Page 3

Slide Evaluations. Circle the most appropriate response

Photo No. 5. Within a building, this crack is encountered in a masonry wall.

- a) Minor damage does not appear to be a concern. Designate "Safe."
- b) Designate "Limited Entry." Mark width and monitor for further movement
- c) Significant structural damage. Designate "Unsafe."
- d) Wall is non-structural partition. Building structure condition cannot be assessed from this damage.

Photo No. 6. While surveying the exterior of a building, this condition is encountered at a concrete wall.

- a) Limited structural damage. Designate building "Limited Entry."
- b) Cordon area in front of wall as "Area Unsafe" AND designate building "Limited Entry."
- c) Significant structural damage. Designate building "Unsafe."
- d) Designate building "Unsafe" AND cordon area in front of wall as "Area Unsafe."

Appendix G

Test Questions for Detailed Evaluators

Name _____

Detailed Evaluation - Exam**Circle the most appropriate response**

1. The following statements are most likely true for an "Ash Fall" NPH Event:
 - a) Evaluations will tend to focus on cracks or sagging of vertical-support elements such as beams, rather than cracking or out-of-plumbness of walls.
 - b) Evaluations of ash fall conditions may need to consider stresses caused by extreme temperatures.
 - c) Rainfall onto buildings which already support 4 to 8 inches of ash depth could quickly cause a moderate NPH event to become serious.
 - d) Answers (a) and © only.
 - e) Answers (a), (b), and (c).

2. A 3-story, moment from building has story heights (from grade) of 15 ft., 13 ft., and 13 ft. Building seismic damage will most like occur:
 - a) at beam-column joints distributed throughout the building.
 - b) at perimeter beam-column connections.
 - c) in upper stories.
 - d) at the lower story.

3. Which statements are true:
 - a) Never walk into a potentially damaged structure without a "buddy" AND having an outside person who knows that you are there.
 - b) A building should be able to resist at least one earthquake after shock about as severe as the main event.
 - c) Adding shoring can cause a structurally unsafe building to become suitable for limited entry or temporary use.
 - d) Answers (a) and (b) only.
 - e) Answers (a), (b), and (c).

4. If the metal siding of a pre-manufactured metal building has become loose or is falling off, it is:
 - a) is an "Area Unsafe" condition.
 - b) is a sign that interior rod bracing may have elongated or that rod bracing connection have deformed.
 - c) it may not be sufficient grounds to classify a structure as "Unsafe."
 - d) All of the above.

5. A concrete wall contains 4 to 6 mostly horizontal cracks of about 1/16" thickness each, and length varying from about 30% to 70% of the wall width.
 - a) The wall has experienced no significant loss of strength.
 - b) The wall retains only about 70% of its initial strength.
 - c) The wall retains only about 30% of its initial strength.
 - d) The wall has experienced a shear failure and may be unsafe.

6. In typical timber post-and-beam construction, areas of probable concern include:
 - a) Splitting of wood at bolted connections.
 - b) Damage of tongue-and-groove board diaphragms.
 - c) Building separation at re-entrant corners.
 - d) All of the above.

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